



PRELIMINARY FOUNDATION INVESTIGATION REPORT

for

ADELAIDE ROAD UNDERPASS

HIGHWAY 402

MTO WEST REGION 59 STRUCTURE REHABILITATIONS

SITE 19-534, CONTRACT 3

GWP 3075-11-00

TOWNSHIP OF STRATHROY-CARADOC

MIDDLESEX COUNTY, ONTARIO

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PRELIMINARY FOUNDATION INVESTIGATION REPORT

for
Adelaide Road Underpass
Highway 402
MTO West Region 59 Structure Rehabilitations
Site 19-534, Contract 3
GWP 3075-11-00
Township of Strathroy-Caradoc
Middlesex County, Ontario

1. INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the proposed rehabilitation of the Adelaide Road Underpass on Highway 402 in the Township of Strathroy-Caradoc, Middlesex County, Ontario. The proposed rehabilitation is a part of the assignment for the rehabilitation of 59 structures in MTO West Region along Highways 4, 6, 401, 402 and 403. The study was carried out by Peto MacCallum Ltd. (PML) for MMM Group Limited (MMM) on behalf of the Ministry of Transportation of Ontario (MTO).

A Technical Memorandum, dated January 21, 2015, was also prepared as a part of this assignment and is presented in Appendix 1. The purpose of the technical memorandum was to summarize the subsurface and groundwater conditions based on available Geocres reports for the design project terms of reference and to update the foundation design recommendations provided in the available reports to the Limit State design terminology in conformance with the requirements of the Canadian Highway Bridge Design Code (CHBDC).

The purpose of this report was to summarize the subsurface stratigraphy encountered at the structure site during the present preliminary investigation to supplement geotechnical data for the design of temporary roadway protection.

For subsurface conditions from previous foundation investigations reference should be made to the Technical Memorandum. The present Foundation Investigation and Design Report should be read in conjunction with the Technical Memorandum.

The elevations in this report are expressed in meters.



2. SITE BACKGROUND AND GEOLOGY

The Adelaide Road Underpass on Highway 402 is located in the Township of Strathroy-Caradoc, Middlesex County, Ontario. The site is located about 18.5 km northwest from the Highways 401 and 402 junction and 19.5 km southeast from Strathroy, Ontario.

Topographically, the immediate area of the site is flat and gently sloping toward the valley of the meandering Thames River which is located about 350 m east northeast and 1.9 km in the southwestern direction from the Adelaide Road Underpass and Highway 402. Currently, residential and agricultural areas are present near the project site.

Physiographically, the site is situated in the region known as the Caradoc Sand Plains. The limestone, dolostone or shale bedrock in the area belongs to Hamilton Group of Middle Devonian period.

3. FIELD INVESTIGATION

A total of two boreholes, AR-1 and AR-2, were drilled on October 30, 2014 at the site location. The boreholes were selected by PML and the survey of the boreholes was conducted by MMM. The boreholes were located in the west and east approaches, respectively. The borehole locations are shown in Drawing AR-1.

The two boreholes, AR-1 and AR-2, were drilled through the soil cover to 14.3 and 12.8 m, respectively. The boreholes were advanced using continuous flight solid augers powered by a track mounted CME 55 drill rig, supplied and operated by a specialist drilling contractor. The drilling crew worked under the full-time supervision of a member of our engineering staff.

Representative samples of the soils encountered in the boreholes were recovered at frequent depth intervals. In the boreholes advanced with conventional drill rigs, soil samples were obtained using a split spoon sampler in conjunction with standard penetration tests. Where standard



penetration tests were not carried out, the consistency/compactness of the encountered soils was estimated from manual examination or the rate (ease) of advance of the augers.

The boreholes were backfilled in accordance with the MTO guidelines and MOE regulation 903 for borehole abandonment procedures using a bentonite/cement mixture grout and asphalt patch.

The groundwater conditions at the borehole locations were assessed during drilling by visual examination of the soil, the sampler and drill rods as the samples were retrieved and, when appropriate, by measurement of the water level in the open boreholes.

Soils were identified in the field in accordance with the MTO Soil Classification procedures. Recovered soil samples were returned to our laboratory for detailed visual examination and soil classification. The laboratory test program for the current subsurface investigation comprised the following tests:

- Natural moisture content determinations (15)
- Grain size analyses (7)
- Atterberg limits (2)

The results of the laboratory natural moisture content determinations and grain size analyses are shown on the Record of Borehole sheets AR-1 and AR-2. The grain size distribution charts are presented on Figures AR-GS-1 to AR-GS-4 and the plasticity charts are presented in Figure AR-PC-1.

4. SUBSURFACE CONDITIONS

Reference is made to the appended Record of Boreholes AR-1 and AR-2 sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, standard penetration test N values, grain size distribution and groundwater observations. The boundaries between soil strata have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.



In summary, the subsurface encountered in the two boreholes included surficial pavement structure over 3.0 to 4.3 m thick embankment fill below which compact to very dense, locally loose (in borehole AR-2), silt/sand/silty sand and very stiff clayey silt soil deposits were encountered.

4.1 Pavement Structure

Surficial 60 and 50 mm thick asphaltic concrete over 140 and 150 mm thick gravelly sand was encountered in boreholes AR-1 and AR-2, which extended to elevations 232.6 and 232.1, respectively. Two N values recorded were 50 blows for 5 and 7 cm penetration depths.

4.2 Fill

Fill was encountered below the pavement structure in boreholes AR-1 and AR-2 at 0.2 m, elevation 232.6 and 232.1, which extended to 3.2 and 4.5 m, elevation 229.6 and 227.8. The fill included sand to sandy silt with inclusions of gravel, cobbles and boulders. N values recorded in the fill were 50 blows for 3 to 13 cm penetration depths.

4.3 Sand

A compact 1.3 and 1.5 m thick sand layer was encountered in boreholes AR-1 and AR-2 below the fill layer at 3.2 and 4.5 m, elevation 229.6 and 227.8, respectively. The sand layer extended to 4.5 and 6.0 m, elevation 228.3 and 226.3 in boreholes AR-1 and AR-2, respectively. Clayey silt seams were encountered within the sand deposit in borehole AR-1. Three N values recorded were 11, 14 and 15.

Grain size distribution results of selected sand samples are presented in Figure AR-GS-1. The samples included 4 and 6% clay, 15 and 22% silt, 71 and 76% sand and 1 and 5% gravel sized particles. Three moisture content determinations were 12, 14 and 15%.



4.4 Silt

A layer of 4.5 m thick silt deposit was encountered below the upper sand layer in boreholes AR-1 and AR-2 at 4.5 and 6.0 m, elevation 228.3 and 226.3, respectively and extended to 9.0 and 10.5 m, elevation 223.8 and 221.8. Generally, the compactness of the silt was dense to very dense; however, a local loose zone (N value of 9) in borehole AR-2 was encountered at 9.1 m, elevation 223.2. N values recorded were 32, 33, and a range between 50 to 88 blows for 10 to 28 cm penetration depths.

A grain size distribution result of a silt sample is presented in Figure AR-GS-2. The sample contained 7% clay, 90% silt and 3% sand sized particles. Moisture content determinations of the silt samples ranged between 19 and 23%.

Further, a grain size distribution result of a silt sample containing clayey silt seams is presented in Figure AR-GS-3 and the corresponding plasticity chart is presented in Figure AR-PC-1. The sample contained 32% clay, 55% silt and 13% sand sized particles. The Atterberg liquid and plastic limits obtained were 23 and 14, respectively. The plasticity index value was 9. Moisture content of the sample was 20%.

4.5 Clayey Silt

A 2.0 m thick very stiff clayey silt deposit was encountered below the silt layer in borehole AR-1 at 9.0 m, elevation 223.8 which extended to 11.0 m, elevation 221.8. One N value recorded was 30. A penetrometer test result obtained a shear strength value of 100 kPa.

A grain size distribution chart of a selected clayey silt sample is presented in Figure AR-GS-3 and the corresponding plasticity chart is presented in Figure AR-PC-1. The sample contained 36% clay, 48% silt and 16% sand sized particles. The Atterberg liquid and plastic limits obtained were 27 and 15, respectively. The plasticity index value was 12. One moisture content determination of the clayey silt sample was 17%.



4.6 Lower Sand

A dense to compact sand deposit was encountered below the silt layer in borehole AR-2 at 10.5 m, elevation 221.8. The sand deposit extended to the termination depth of the borehole at 12.8 m, elevation 219.5. Two N values recorded were 34 and 28.

A grain size distribution chart of a selected sand sample is presented in Figure AR-GS-1. The sample contained 8% clay, 20% silt and 72% sand sized particles. Two moisture content determinations were 19 and 21%.

4.7 Silty sand

A compact silty sand layer deposit was encountered in borehole AR-1 below the clayey silt layer at 11.0 m, elevation 221.8, which extended to the termination depth of the borehole at 14.3 m, elevation 218.5. Three N values recorded in sequence were 28, 14 and 20.

A grain size distribution result of a selected sample is presented in Figure AR-GS-4. The sample included 2% clay, 34% silt and 64% sand sized particles. Moisture content determinations ranged between 20 and 21%.

4.8 Groundwater

Groundwater was encountered in boreholes AR-1 and AR-2 at 4.6 m, elevation 228.2 and 227.7, respectively, during augering. Upon completion of augering, groundwater was encountered in borehole AR-1 at 8.5 m, elevation 224.3 and in borehole AR-2 at 7.9 m, 224.4.

It should be noted that the groundwater level is subjected to fluctuations due to seasonal and rainfall patterns.

5. MISCELLANEOUS

Mr. S. Aziz carried out the field investigation for this study under the supervision of Mr. N. Rahman, P.Eng. Fisher Environmental Ltd. supplied the drilling equipment for the subsurface exploration. The laboratory testing of the selected samples was carried out in the PML laboratory in Toronto.



6. CLOSURE

This preliminary report was prepared by Mr. N. Rahman, P.Eng and was reviewed by Mr. R. Ng, PhD, P.Eng., Senior Project Engineer. Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact conducted an independent review of the report.

Yours very truly,

Peto MacCallum Ltd.



Nazibur Rahman, P.Eng.
Project Engineer, Geotechnical Services



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Senior Project Engineer



Brian R. Gray, M.Eng, P.Eng.
MTO Designated Principal Contact

NR/RN/BRG:nr-mi-jk

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

COMPOSITION: SECONDARY SOIL COMPONENTS ARE DESCRIBED ON THE BASIS OF PERCENTAGE BY MASS OF THE WHOLE SAMPLE AS FOLLOWS:

PERCENT BY MASS	0 - 10	10 - 20	20 - 30	30 - 40	> 40
	TRACE	SOME	WITH	ADJECTIVE (SILTY)	AND (AND SILT)

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE
F V	FIELD VANE		

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_i	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	n	1, %	POROSITY	e_{max}	1, %	VOID RATIO IN LOOSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	w	1, %	WATER CONTENT	e_{min}	1, %	VOID RATIO IN DENSEST STATE
ρ_w	kg/m ³	DENSITY OF WATER	S_r	%	DEGREE OF SATURATION	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
γ_w	kN/m ³	UNIT WEIGHT OF WATER	w_L	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_p	%	PLASTIC LIMIT	D_n	mm	n PERCENT - DIAMETER
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_s	%	SHRINKAGE LIMIT	C_u	1	UNIFORMITY COEFFICIENT
ρ_d	kg/m ³	DENSITY OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	q	m ³ /s	RATE OF DISCHARGE
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL				i	1	HYDRAULIC GRADIENT
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	j	kN/m ³	SEEPAGE FORCE
e	1, %	VOID RATIO	WTPL		WETTER THAN PLASTIC LIMIT			

RECORD OF BOREHOLE No. AR-1

1 of 2

METRIC

G.W.P. 3075-11-00 **LOCATION** Coords: 4 752 095.7 N; 391 181.6 E **ORIGINATED BY** S.A.
DIST London **HWY** 402 **BOREHOLE TYPE** Continuous Flight Solid Stem Augers **COMPILED BY** N.R.
DATUM Geodetic **DATE** October 30, 2014 **CHECKED BY** B.R.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa									
							20 40 60 80 100									
							20 40 60 80 100									
232.8	Ground Surface		1	SS	50/5cm											
232.6 0.2	60mm asphalt over 140mm gravelly sand (PAVEMENT FILL) Sand, with gravel cobble and boulders Very dense Brown Moist (FILL)		2	SS	50/10cm											
			3	SS	50/13cm											
			4	SS	50/13cm											
229.6 3.2	Sand, some silt trace clay, trace gravel Compact Brown Moist Clayey silt seams		5	SS	14											5 76 15 4
			5A	SS	15											
228.3 4.5	Silt trace sand, trace clay Dense to Brown Wet very dense		6	SS	50/13cm											
			6A	SS	33											0 3 90 7
			7	SS	32											
			8	SS	80/10cm											
223.8 9.0	Clayey silt, some sand Very stiff Brown Moist		9	SS	30											0 16 48 36
221.8 11.0	Silty sand, trace clay compact Brown Wet		10	SS	28											
			11	SS	14											0 64 34 2
			12	SS	20											
218.5 14.3	End of borehole															

Cont'd

RECORD OF BOREHOLE No. AR-1

2 of 2

METRIC

G.W.P.	3075-11-00	LOCATION	Coords: 4 752 095.7 N; 391 181.6 E	ORIGINATED BY	S.A.
DIST	London	HWY	402	BOREHOLE TYPE	Continuous Flight Solid Stem Augers
DATUM	Geodetic	DATE	October 30, 2014	CHECKED BY	B.R.G.
COMPILED BY N.R.					

[illegible]

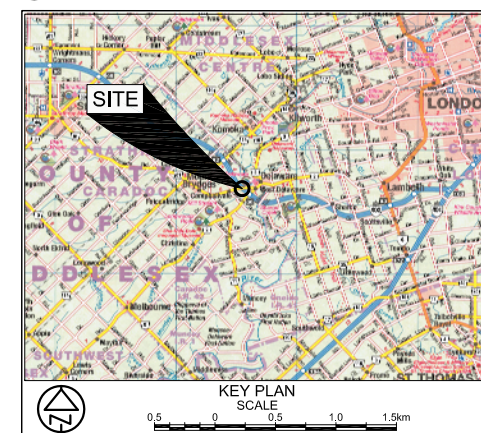
RECORD OF BOREHOLE No. AR-2

1 of 1

METRIC

G.W.P.	3075-11-00	LOCATION	Coords: 4 752 114.0 N; 391 301.2 E	ORIGINATED BY	S.A.
DIST	London	HWY	402	BOREHOLE TYPE	Continuous Flight Solid Stem Augers
DATUM	Geodetic	DATE	October 30, 2014	COMPILED BY	N.R.
				CHECKED BY	B.R.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)					
								○ UNCONFINED		+ FIELD VANE		● QUICK TRIAXIAL						× LAB VANE					
								20	40	60	80	100						20	40	60	20	40	60
232.3	Ground Surface		1	SS	50/7cm																		
232.1 0.2	50mm asphalt over 150mm gravelly sand (PAVEMENT FILL) --- Sandy silt, with gravel cobble and boulders Very dense Brown Moist (FILL)		2	SS	50/3cm																		
			3	SS	50/8cm																		
			4	SS	50/10cm																		
			5	SS	50/3cm																		
227.8 4.5	Sand, with silt trace clay, trace gravel Compact Brown Wet		6	SS	11																		
226.3 6.0	Silt trace sand, trace clay Very dense Brown Wet		7	SS	88/28cm																		
	Clayey silt seams some sand		8	SS	50/13cm																		
	some to with sand trace clay																						
			9	SS	9																		
221.8 10.5	Sand some to with silt Dense to Brown/ Wet compact grey		10	SS	34																		



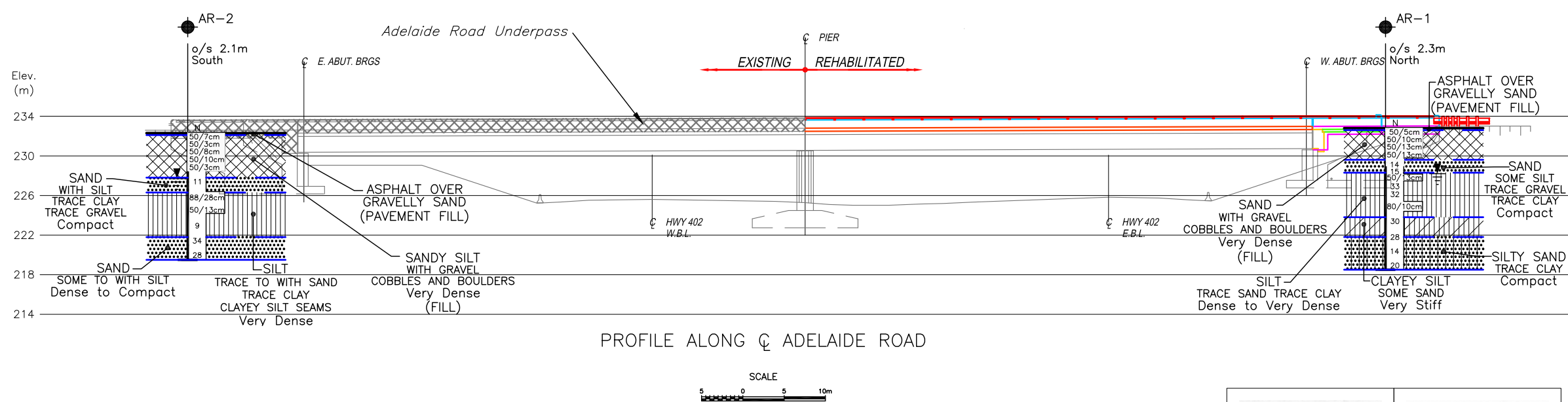
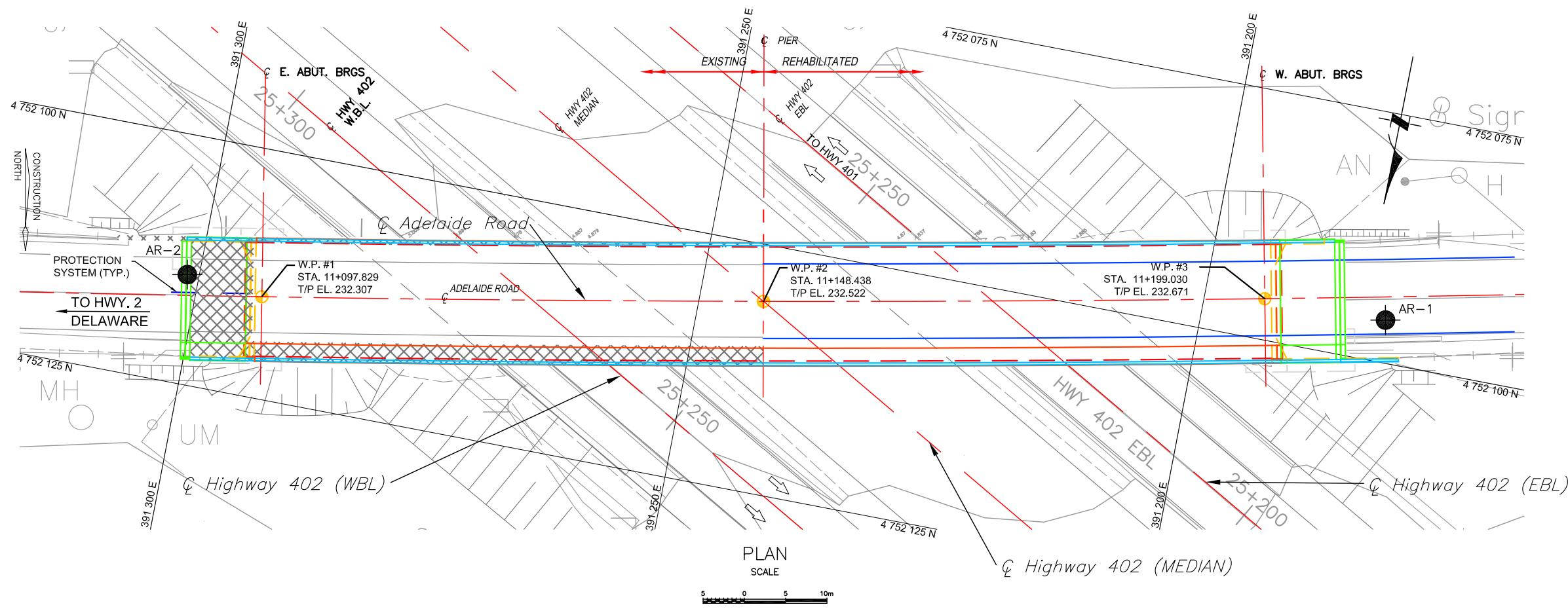
LEGEND			
	Borehole		
	Borehole and cone		
	Cone penetration test		
N	Blows/0.3m (Std. Pen Test, 475 J/blow)		
CONE	Blows/0.3m (60° Cone, 475 J/blow)		
	W L at time of investigation Oct. 2014		
	Head		
	ARTESIAN WATER		
	Encountered		
	PIEZOMETER		

BH No	ELEVATION	COORDINATES	
		NORTHINGS	EASTINGS
AR-1	232.8	4 752 095.7	391 181.6
AR-2	232.3	4 752 114.0	391 301.2

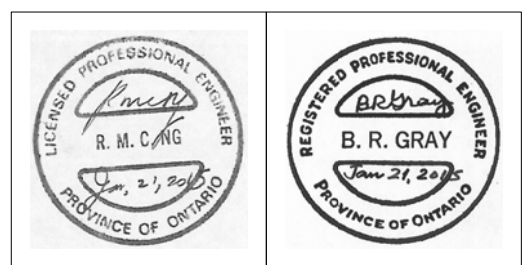
— NOTE —
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION

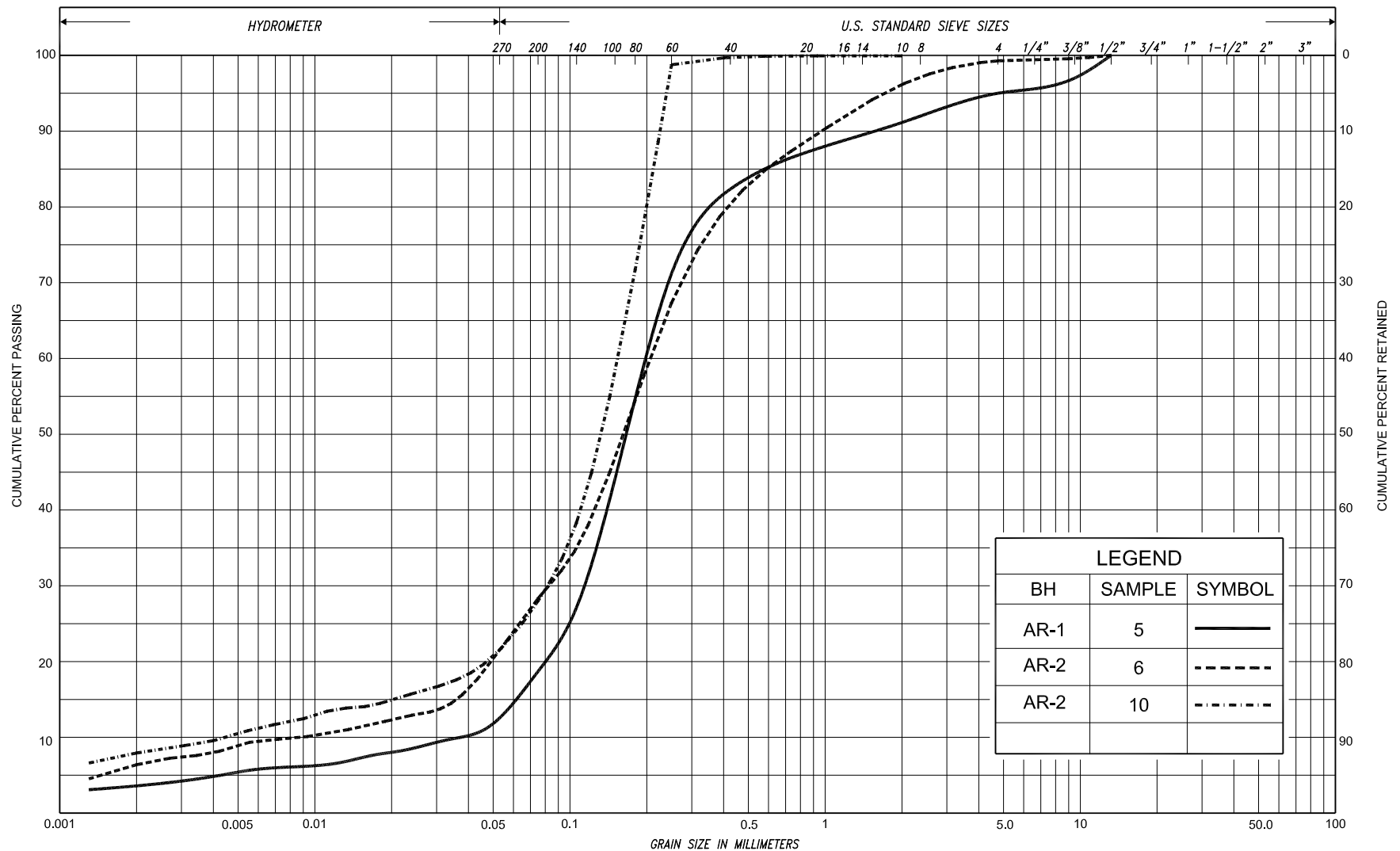
Geocres No. 40114-154			
HWY No	402	CHECKED	NR
SUBMD	NA	DATE	JAN. 21, 2015
DRAWN	NA	APPROVED	BRG
DIST	LONDON	SITE	19-534
		DWG	AR-1



- NOTES:
- THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH RECORD OF BOREHOLES AND REPORT
 - THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
 - DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.



Reference Composite of MMM Drawings:
S381 3001-311-001.GA.dwg; -311-002XG.dwg; _XB01_Adelaide.dwg
and 381 3001-313-001XG.dwg dated November 2014



SILT & CLAY				FINE		MEDIUM		COARSE	GRAVEL		COBBLES	UNIFIED		
CLAY	FINE		MEDIUM	COARSE	FINE		MEDIUM	COARSE	GRAVEL		COBBLES	M.I.T.		
	SILT				SAND				GRAVEL			U.S. BUREAU		
CLAY		SILT		V. FINE	FINE	MED.	COARSE	GRAVEL						
				SAND										



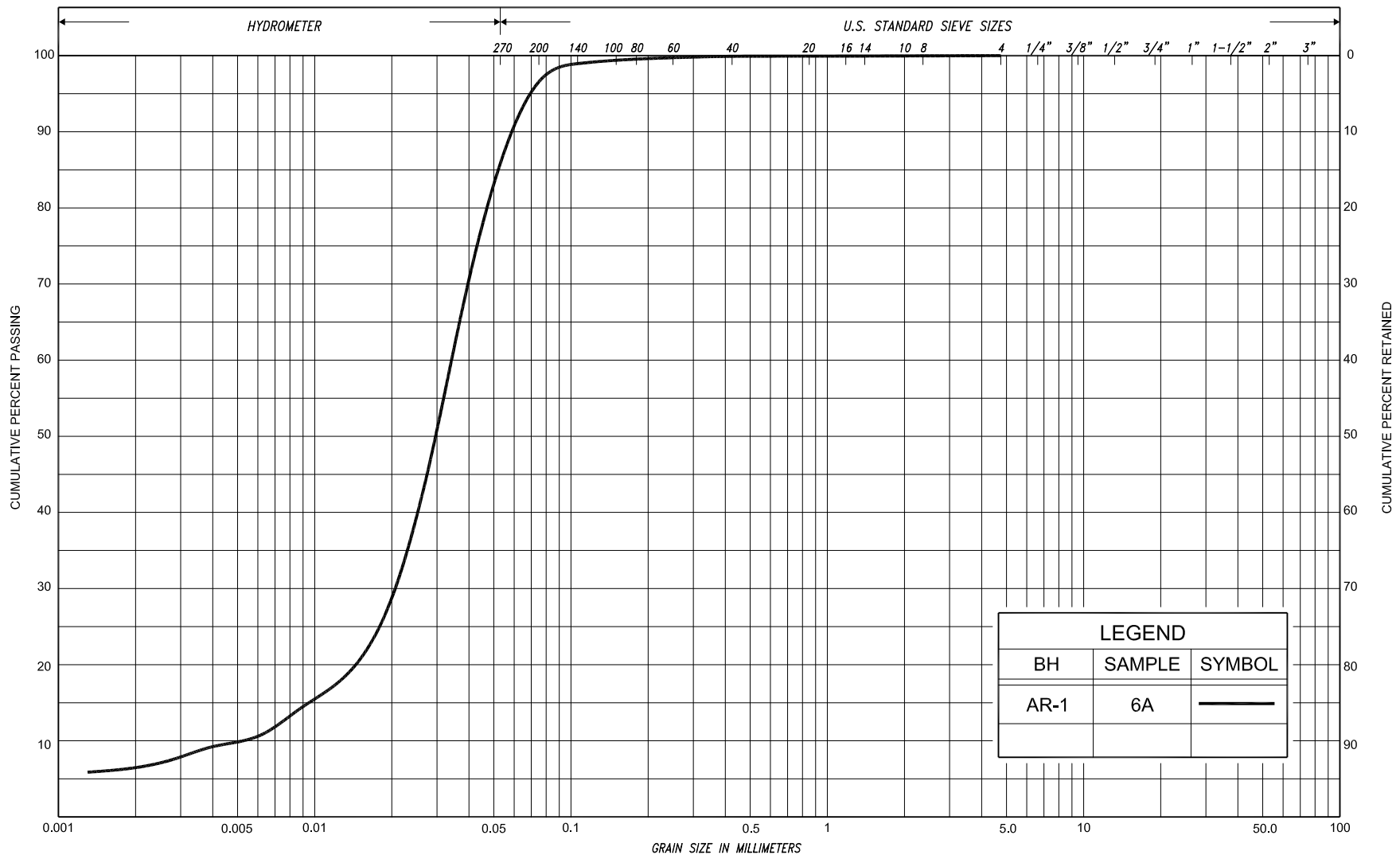
GRAIN SIZE DISTRIBUTION

SAND, some to with silt, trace clay, trace gravel

FIG No. AR-GS-1

HWY: 402

G.W.P. No. 3075-11-00



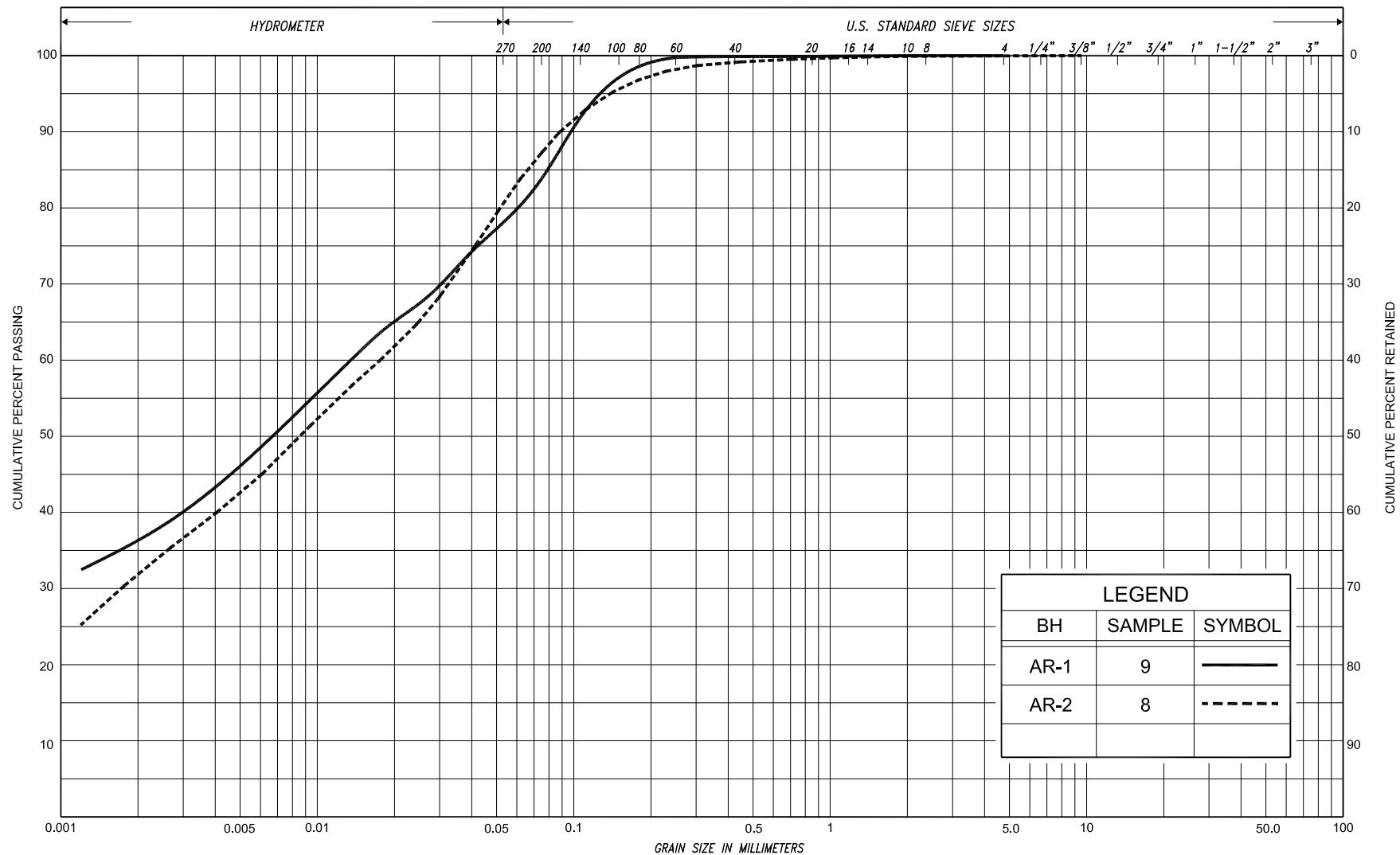
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					SAND													
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	SILT																	
CLAY		SILT			V. FINE	FINE	MED.	COARSE		GRAVEL								U.S. BUREAU
					SAND													



GRAIN SIZE DISTRIBUTION

SILT, trace sand, trace clay

FIG No. AR-GS-2
 HWY: 402
 G.W.P. No. 3075-11-00



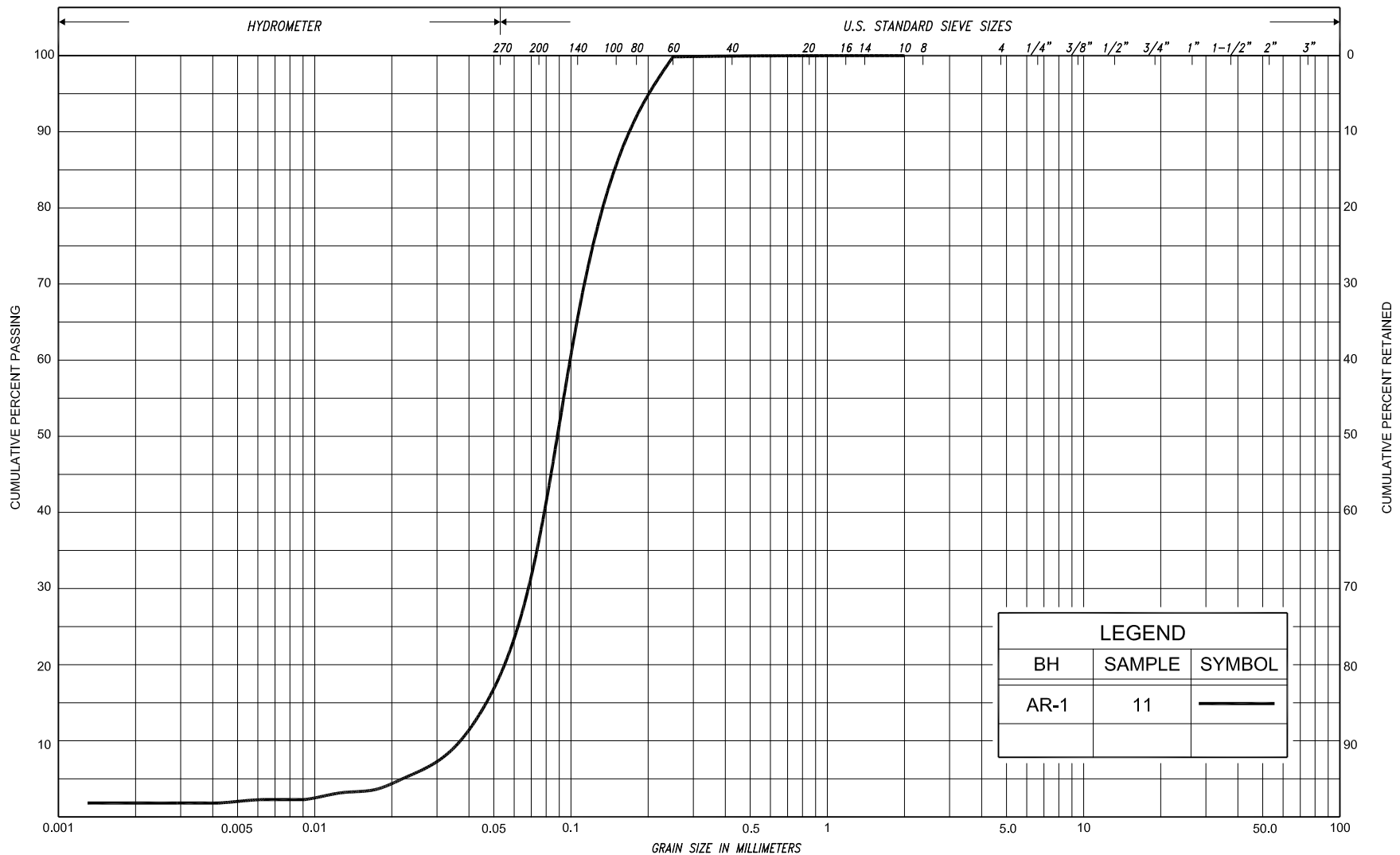
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					SAND											
CLAY	FINE		MEDIUM		COARSE	FINE		MEDIUM		COARSE		GRAVEL			COBBLES	M.I.T.
	SILT					SAND										
CLAY			SILT			V. FINE	FINE	MED.	COARSE		GRAVEL					U.S. BUREAU
					SAND											



GRAIN SIZE DISTRIBUTION

CLAYEY SILT, some sand (CL)

FIG No. AR-GS-3
 HWY: 402
 G.W.P. No. 3075-11-00



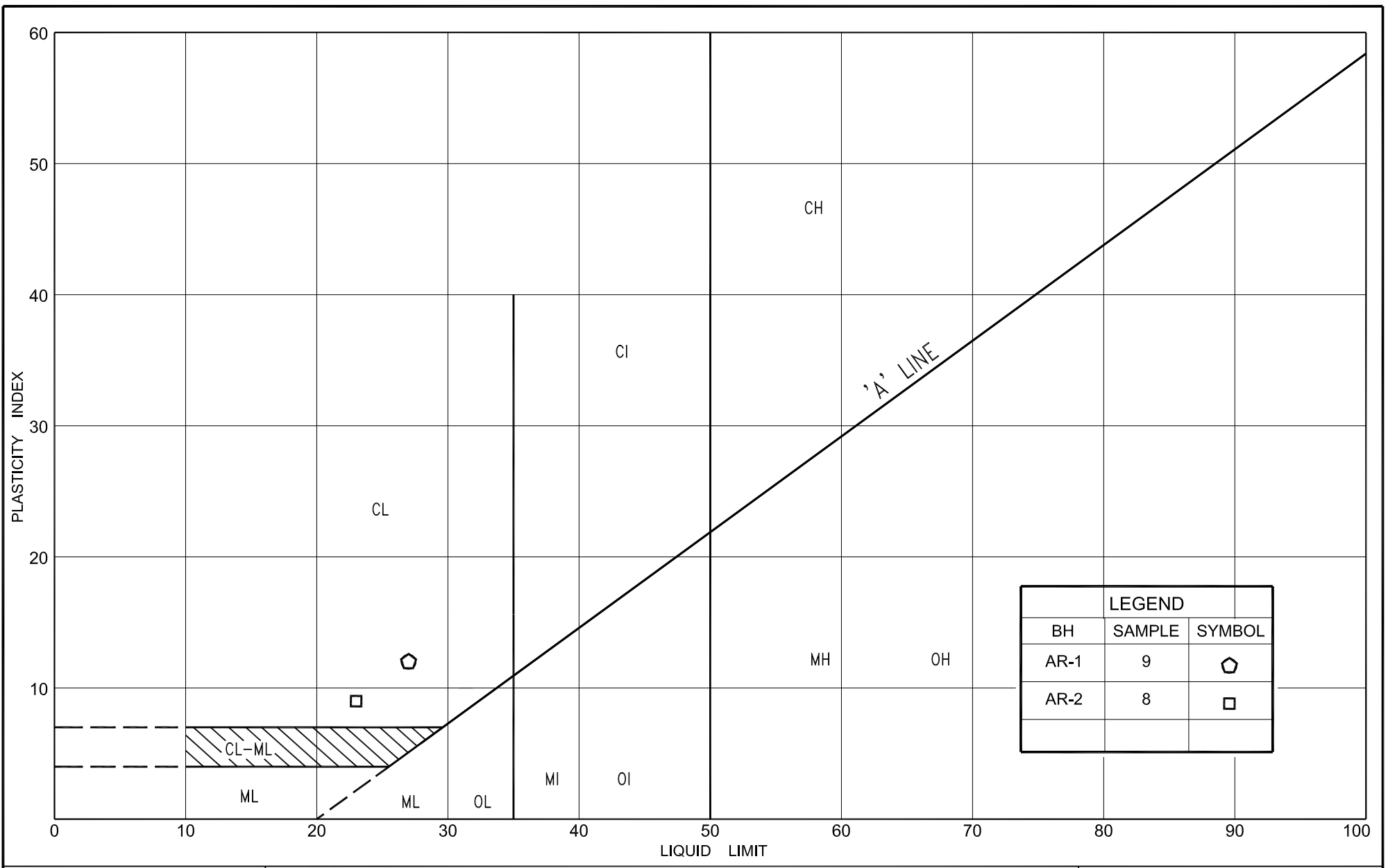
SILT & CLAY				FINE SAND			COARSE SAND	GRAVEL	COBBLES	UNIFIED
CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		GRAVEL	COBBLES	M.I.T.
CLAY	SILT			V. FINE	FINE	MED.	COARSE	GRAVEL		U.S. BUREAU



GRAIN SIZE DISTRIBUTION

SILTY SAND, trace clay

FIG No. AR-GS-4
 HWY: 402
 G.W.P. No. 3075-11-00



PLASTICITY CHART CLAYEY SILT, some sand (CL)

FIG No. AR-PC-1

HWY: 402

G.W.P. No. 3075-11-00



APPENDIX 1

TECHNICAL MEMORANDUM



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Table 1 – List of Standard Specifications

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– General Layout, Highway 81 Underpass

Appendix B – Site Photographs

FOUNDATION TECHNICAL MEMORANDUM

For

Adelaide Road Underpass, Highway 402
MTO West Region 59 Structure Rehabilitations
Site 19-534, Contract 3,
GWP 3075-11-00
Township of Strathroy-Caradoc
Middlesex County, Ontario

1. INTRODUCTION

The Foundation Engineering Services for the present project involve the detail foundation investigation and design for rehabilitation of the 59 structures in MTO West Region along Highways 4, 6, 401, 402 and 403. Ten (10) Group Work Projects (GWP's) are contemplated to be completed between 2014 and 2020.

This technical memorandum summarizes the factual results of the geotechnical data based on the review and compilation of existing subsurface information from relevant reports in MTO GEOCRES Library for the Highway 402 Adelaide Road Underpass in the Township of Strathroy-Caradoc, Middlesex County, Ontario. The Foundation Engineering recommendations from the original foundation reports are summarized with reference to the "Canadian Highway Bridge Design Code" (CHBDC) and follow in general the "Guidelines for Professional Engineers providing Geotechnical Engineering Services".

From the Minutes of Meeting Report, dated May 5, 2014, it is understood that the rehabilitation of the underpass structure will include replacement of the existing concrete barrier walls and conversion to semi-integral abutments. It will be constructed in two stages and roadway protection will be required during conversion to semi-integral abutments.

The purpose of this technical memorandum is to summarize the subsurface and groundwater conditions and foundation recommendations based on available reports for the design project team's reference.



2. PROJECT SITE BACKGROUND AND GEOLOGY

The Adelaide Road Underpass on Highway 402 is located in the Township of Strathroy-Caradoc, Middlesex County, Ontario. A key plan is shown in Figure 1. The site is located about 18.5 km northwest from the Highways 401 and 402 junction and 19.5 km southeast from Strathroy, Ontario.

Topographically, the immediate area of the site is flat and gently sloping toward the valley of the meandering Thames River which is located about 350 m east-northeast and 1.9 km in the southwestern direction from the Adelaide Road Underpass and Highway 402. Currently, residential and agricultural areas are present near the project site.

Physiographically, the site is situated in the region known as the Caradoc Sand Plains. The limestone, dolostone or shale bedrock in the area belongs to Hamilton Group of Middle Devonian period.

3. SOURCE OF INFORMATION

The following reports, documents and maps were available for review and information for the Adelaide Road Underpass, and are appended in Appendix A. Reference 1 below represents the foundation investigation report for the final bridge alignment.

1. Foundation Investigation Report, for Highway 81 Underpass, W.P. 40-66-03, Site 19-534, 0.7 miles west of Junction of Highway 2 and Highway 81, District 2, London, Ontario by Soil Mechanics Section Geotechnical Office, Ministry of Transportation and Communications, Ontario, dated April 8, 1976. GEOCRES No. 40I14-104. (Reference 1)
2. General Layout Highway 81 Underpass, 0.5 mile West of Highway 2, District 2, London, W.P. No. 40-66-03, CONT. No. 80-77, dated January 1978. (Reference 2)



4. SITE RECONNAISSANCE

As part of the current foundation engineering assessment study, a site reconnaissance of the Highway 402 Adelaide Road Underpass was carried out on February 19, 2014. A photographic record of the site visit is attached in Appendix B.

The site photographs present the current conditions of east and west abutments and center pier of Adelaide Road Underpass including appearance of structure, visual slope stability, soil erosion, and vegetation on the slopes.

The adjacent north and south slopes of the east and west abutments were covered with snow during the site reconnaissance. The limited exposed slope surface showed no sign of erosion (Photograph 4). The front slopes away from the abutments were covered with rip-rap. No indication of erosion was observed on the faces of the slopes. Surface wall cracks were observed on the east abutment wall (Photograph 3). The front faces away from the abutments were sloped at approximately 2H:1V. Further, the flat grounds in front of the abutments were exposed and slight erosion was observed (Photographs 3 and 8).

No major deteriorations were observed on the wingwalls of the east abutment (Photographs 4 and 5) and the west abutment (Photographs 9 and 10). Surface cracks were observed on the wingwalls and abutment walls. The slopes away from the wingwalls were in a visibly surficial stable condition during the time of the visit.

Surficial cracks were observed on the centre pier surface and slight erosion of soil around the pier was observed (Photograph 6).

Based on the General Arrangement Drawing (Reference 2), a 150 mm diameter perforated CSP was placed behind the east and west abutment walls; however, the CSPs could not be assessed during the site reconnaissance. Weep drains were observed in the abutment walls and wingwalls (Photographs 2, 3, 7 and 8). No water ponding was observed in front and adjacent areas of the abutment walls indicating that the weep holes are functioning.



5. PREVIOUS INVESTIGATION AND SUMMARIZED SUBSURFACE CONDITIONS

A foundation investigation report was prepared by Soil Mechanics Section Geotechnical Office, Ministry of Transportation and Communications, Ontario dated April 8, 1976. The purpose of the investigation was to establish the subsoil and groundwater conditions at the proposed underpass location.

The field work included four boreholes (boreholes 1, 3, 7 and 8); three of them accompanied with dynamic cone penetration tests (DCPTs). The field investigation was carried out during the period of December 1 to 12, 1975. The four boreholes were drilled to 12.6 to 37.0 m (elevation 194.5 to 219.3). In addition, four shallow holes (test holes 2, 4, 5 and 6) were drilled to depths of 1.5 to 6.1 m for groundwater level observation.

The boreholes were advanced using continuous flight augers with a track mounted drill rig. BX casing and washboring method was employed when borehole advancement was prevented due to cave-in of soils. Soil samples were recovered from the boreholes using the standard penetration test method. Undisturbed soil samples were recovered using Shelby tubes, which were pushed into the soil hydraulically.

The borehole and the shallow hole locations and elevations were surveyed by personnel from the Southwestern Region Engineering Surveys. All the borings completed for the project were shown on Drawing 406603-A.

Samples were visually examined in the field and subsequently in the laboratory. Selected samples were subjected to laboratory tests to determine the physical properties of the various soil types. The results of the field and laboratory tests were presented in the Record of Borehole Sheets, appended in Appendix A of the original report (Reference 1).

The subsoil conditions have been established only at the borehole location and were found to differ from borehole to borehole in vertical and horizontal conditions.



5.1 West Abutment

Boreholes 1 and 3 were investigated at the west abutment of the underpass structure.

5.1.1 Sand

A surficial 2.5 and 2.6 m thick very loose to compact sand deposit was encountered in boreholes 1 and 3 and extended to elevation 229.0 and 229.1, respectively. N values recorded ranged from 3 to 28.

The grain size distribution results of the sand samples composed of 7 to 28% silt and clay, 67 to 93% sand, and 0 to 4% gravel sized particles. Moisture content determinations obtained were between 13 and 22%.

5.1.2 Upper Clayey Silt

An upper hard 1.9 and 2.0 m thick clayey silt layer was encountered below the sand deposit in boreholes 1 and 3 at 2.5 and 2.6 m, elevation 229.0 and 229.1. The clayey layer extended to 4.4 and 4.6 m, elevation 227.1, in boreholes 1 and 3, respectively. Seams and pockets of sand were encountered within the clayey silt layer. N values recorded were between 31 and 47.

The grain size distribution results of the clayey samples composed of 20 and 25% clay, 68 and 79% silt and 1 and 7% sand sized particles. The Atterberg liquid limits ranged from 20 to 30 and the plastic limits ranged from 12 to 15 with plasticity index values were between 5 and 15. Moisture content determinations obtained were between 14 and 18%.

A grain size distribution result of a sand sample encountered within the clayey silt composed 3% silt and clay, 92% sand and 5% gravel sized particles. The moisture of the sand was 13%.



5.1.3 Silt

A localized 1.4 m thick dense to compact silt layer was encountered below the clayey silt at 4.6 m, elevation 227.1 in borehole 3 and extended to 5.9 m, elevation 225.7. Two N values recorded were 26 and 44.

A grain size distribution result of a silt sample composed of 11% clay, 81% silt and 8% sand sized particles. Two moisture content determinations obtained were 18 and 22%.

5.1.4 Silty Sand

In borehole 1, a 10.4 m thick compact to dense silty sand layer was encountered below the clayey silt at 4.4 m, elevation 227.1 and extended to 14.8 m, elevation 216.7. N values recorded between 11 and 41. Within the silty sand layers, irregular layers, seams and pockets of clayey silt were encountered.

The grain size distribution results of two selected silty sand samples composed 18 and 30% clay, 38 and 45% silt, and 25 and 44% sand sized particles. Further, the Atterberg liquid limits of two samples from silty sand layer containing clayey silt (in borehole 1), were 22 and 25 and the plastic limits were 12 and 14. The plastic indices were 10 and 11. Moisture content determinations ranged between 13 and 20%.

5.1.5 Lower Clayey Silt

Below the silty sand in borehole 1 and silt in borehole 3, a lower stiff to hard clayey silt layer was encountered at 14.8 and 5.9 m, elevation 216.7 and 225.7, respectively. N values of 14 to 94 were recorded. The lower clayey silt stratum extended to 35.1 and 8.2 m, elevation 196.4 and 223.4, in boreholes 1 and 3, respectively. Irregular layers, seams and pockets of sand and silt were encountered also within the lower clayey silt layer.

Atterberg liquid limits for selected lower clayey silt samples were between 23 and 24 and the plastic limits were between 11 and 15. The plasticity index values ranged from 10 and 14. The moisture content of the lower clayey silt samples ranged between 18 and 20%.



5.1.6 Silty Sand to Sandy Silt

The lower clayey silt layer was underlain by compact to dense silty sand to sandy silt stratum in borehole 3 at 8.2 m, elevation 223.4, and extended to the termination depth of 14.2 m, elevation 217.5. N values of 10 to 40 were recorded. Seams and pockets of clay were encountered.

Grain size distribution test results of two samples composed 6 and 20% clay, 35 and 48% silt, 32 and 59% sand sized particles. Moisture contents ranged between 14 and 22%.

5.1.7 Glacial Till

A 1.9 m thick dense glacial till deposit was encountered below the lower clayey silt deposit in borehole 1 at 35.1 m, elevation 196.4 and extended to the borehole termination depth of 37.0 m, elevation 194.5. One N value of 36 was recorded. The glacial till encountered was a mixture of gravel, sand, silt and clay sized particles in variable proportions.

5.2 Centre Pier

Borehole 7 was drilled adjacent to the center pier of the structure.

5.2.1 Sand

Surficial 3.0 m thick loose to very dense sand was encountered in borehole 7 that extended to elevation 228.9. N values between 7 and 111 were recorded. One moisture content determination obtained was 21%.

5.2.2 Clayey Silt

Below the surficial sand layer, a 9.6 m thick of hard to stiff clayey silt deposit was encountered and extended to the borehole termination depth 12.6 m, elevation 219.3. N values of 14 to 39 were recorded.



Grain size distribution test results of selected samples composed 10 to 49% clay, 46 and 67% silt and 5 to 37% sand sized particles. The Atterberg liquid limits for the clayey silt samples ranged from 18 to 32 and plastic limits ranged from 10 to 17. The plasticity index values ranged from 6 to 20. The moisture content determinations ranged from 15 to 23%. Unit weight of the silty clay samples obtained varied from 19.6 to 21.4 kN/m³.

5.3 East Abutment

Borehole 8 was located at the east abutment of the underpass structure.

5.3.1 Sand

A surficial 2.9 m thick compact to dense sand layer was encountered in borehole 8 and extended to elevation 229.4. N values of 11 to 41 were recorded. Gravel was encountered in the sand layer.

Grain size distribution test results of two sand samples composed of 13 and 14% silt and clay, and 86 and 87% sand sized particles. Two moisture content determinations were 8 and 18%.

A grain size distribution test result of a sand with gravel sample composed 6% silt and clay, 73% sand and 21% gravel sized particles. The moisture content determination of the sample was 11%.

5.3.2 Upper Clayey Silt

An upper 6.9 m thick stiff to hard clayey silt layer was encountered below surficial sand layer at 2.9 m, elevation 229.4 and extended to 9.8 m, elevation 222.6. N values recorded ranged from 15 to 41. Irregular layers seam and pockets of sand and silt were encountered within the clayey silt layer.

Grain size distribution results of selected clayey silt samples composed 26 to 34% clay, 55 to 70% silt, and 4 to 12% sand sized particles. The Atterberg liquid limits were between 20 and 23 and the plastic limits were between 11 and 12. The plasticity index values obtained were between 8 and 11. Moisture content determinations of the upper clayey samples ranged from 14 to 20%.



5.3.3 Silty Sand to Sandy Silt

A 2.3 m thick compact to dense silty sand to sandy silt deposit was contacted below the clayey silt layer at 9.8 m, elevation 222.6, and extended to 12.0 m, elevation 220.3. Three N values recorded were between 23 and 32.

Grain size distribution results of two selected samples composed 17 and 21% clay, 32 and 65% silt, and 18 and 47% sand sized particles. Two moisture content determinations were 19%.

5.3.4 Middle Clayey Silt

A middle 2.0 m thick very stiff to hard clayey silt layer was contacted below the silty sand to sandy silt deposit at 12.0 m, elevation 220.3 and extended to 14.0 m, elevation 218.3. Two N values recorded were 15 and 71.

An Atterberg liquid limit of a sample was 27 and the plastic limit was 13. The plasticity index was 14. One moisture content determination was 21%.

5.3.5 Silt

The middle clayey silt deposit was underlain at 14.0 m, elevation 218.3, by a 6.1 m thick compact to dense silt layer that extended to 20.1 m, elevation 212.2. N values recorded between 22 and 32. Irregular layers of clayey silt were encountered in the silt layer.

Grain size distribution results of two selected samples composed 21 and 25% clay, 66 and 72% silt, and 7 and 9% sand sized particles. Two moisture content determinations were 18 and 20%.

5.3.6 Lower Clayey Silt

A 14.0 m thick lower very stiff to hard clayey silt layer was contacted below the silt layer at 20.1 m, elevation 212.2 and extended to 34.1 m, elevation 198.2. N values recorded in the layer ranged from 29 to 87.



One grain size distribution result of a selected sample composed 37% clay and 63% silt sized particles. Atterberg limit and plastic limits obtained for a selected sample were 18 and 12, respectively. The plasticity index value obtained was 6. Two moisture content determinations of selected lower clayey silt samples were 24 and 25%.

5.3.7 Glacial Till

A 1.6 m thick very dense glacial till layer was encountered below the lower clayey silt layer at 34.1 m, elevation 198.2 and extended to the borehole termination depth at 35.7 m, elevation 196.7. One N value of 167 was recorded.

5.4 Groundwater

Groundwater was observed in the boreholes and shallow boreholes during the field investigation at 0.8 to 1.9 m, elevation 230.4 to 230.8. The water level observation was carried out during a relatively dry period and that higher levels may prevail in the spring period. The non cohesive layers interbedded within the cohesive deposits were water bearing and that seepage from these layers were anticipated.

A Geotechnical Report prepared by Hydrology Consultants Ltd. Indicated that the bedrock surface is located some 68.6 m (225 ft), elevation 163.0 (535 ± ft) below ground level in the site area (in Reference 1).

6. FOUNDATION

6.1 Previous Foundation Discussion and Recommendations

It was proposed to build a two span underpass structure either with 49.8 – 49.8 m (163.4 – 163.4 ft) or 57.4 – 55.9 m (188.4 – 183.4 ft) long spans at this location. The report noted that the profile grade of Highway 402 was at elevation 225.5 (740 ± ft) about 6.7 m lower than the profile grade of Adelaide Road at the time of investigation. The subsurface soils encountered at the borehole locations varied



in type, consistency and extent. The observed groundwater level was 0.8 to 1.9 below existing ground level in time of investigation, elevation 230.4 to 230.8.

6.1.1 Foundation

The report considered two types of foundation for the underpass structure - spread footings and piles.

6.1.1.1 Spread Footings

Based on the subsurface factual data, it was recommended that the spread footings of the abutments may be founded at elevation $228.0 \pm$ (748 \pm ft) with a safe design load of 2.5 tsf (240 kPa) for design purposes of the structure. At this elevation, the subsoil consists of very stiff to hard clayey silt layer with interbedded sand and silts.

The laboratory undrained shear strength for the cohesive portion ranged between 95.7 to 191.5 kPa (2000 to 4000 psf). However, it was anticipated that the base of the footing excavations could contain water bearing sand and silt layer, which are considered sensitive to disturbance. The report anticipated that the dewatering in addition to decrease in bearing capacity could present a problem at the site. Hence, it was recommended that interlocking sheet piles be installed to a depth equal to the hydrostatic head existing above the footing excavation base at the time of the construction.

The report also recommended that the underside of the footing be placed within the cohesive portion of the subsoil and that all non-cohesive materials be removed from the excavation base.

Further, it was recommended that the front face of the abutment footing should not be placed closer than 3.0 m from the forward slope surface. For computation of sliding resistance, an adhesion value of 143.5 kPa or a coefficient of 0.35 was recommended, which ever produced a lesser resisting force, assumed to apply between bases of footings and the underlying soil at the foundation level.

At the pier location, similar subsurface was encountered and the report recommended to found the pier footing at elevation $223.7 \pm$ (734 ft. \pm) with safe design load of 2.0 tsf (191.5 kPa) and adhesion



value of 95.7 kPa or a coefficient of 0.35 to be used. The general criteria of abutment footing should apply for the pier footing also.

6.1.1.2 Piles

Based on the report, the entire structure may be supported on two different pile types, timber or H piles.

For #14 timber piles, it was recommended to drive the piles to about 13.7 m (elevations 217.8 to 218.6) into the original ground and use a design load of 25 tons (250 kN) per pile. For the H pile, it was recommended to drive the piles beyond elevation 195.1 (640 ft), 36.4 to 37.2 m, to achieve maximum allowable load. It was recommended to control the pile driving during the construction by employing Hiley Dynamic Pile Driving Formula (MTC Standard SS3-10 and 11).

6.1.2 Frost Protection

The report recommended a frost protection of 1.2 m earth cover for all spread footings and the pile caps of the structure.

6.1.3 Settlement

It was anticipated in the original report that the total settlement should not be more than 25 mm and differential settlement between piers and abutment should be less than 25 mm.

6.1.4 Roadway Cut

At the structure, about 6.7 m deep cut was anticipated for the Highway 402. Based on the profile grade of Highway 402 at the time of the investigation, groundwater level was considered to be about 4.9 to 5.2 m higher than the profile. The water bearing non-cohesive deposits interbedded within the cohesive layers was considered a potential problem since freezing and thawing could cause surficial instability by creating large seepage forces on the face of the slope. To ensure the long term stability of the cut slope the report recommended the following actions should be implemented. First, perforated subdrains should be constructed at the toe of the slope with adequate outlet and the drain



should be protected from frost penetration. Second, a 0.46 m (18 in.) thick granular 'A' blanket should be provided at 2H: 1V over the slope surface on both sides of the proposed cuts.

It was further recommended to protect the slopes against erosion as per MTC practices.

6.1.5 Construction Considerations

In order to minimize the construction problems which may occur due to the observed high groundwater level, the following sequence of construction was suggested:

- a) Excavate for roadway
- b) Construct subdrains and place granular 'A' blanket on slopes
- c) Construct structure

Based on the general layout of Adelaide Road Underpass (Reference 2), the proposed bridge was to be constructed as a two span structure of 50.6-50.6 m (166-166 ft). The foundation of the abutments and the pier was shown to be founded on spread footings. The top of the east abutment footing was shown at approximate elevation 226.9 (744.5 ft) and the top of the west abutment footing was shown at approximate elevation 227.5 (746.5 ft). The top of the pier footing was shown at elevation 224.5 (736.5 ft). Perforated CSP pipes (15 cm or 6 in. diameter) were to be placed behind the abutments to control seepage.

The general layout drawing showed the front slopes of the abutments were to be covered with concrete; however, based on the site reconnaissance it was observed that the slope was covered with rip-rap stones.

6.2 Assessment of Foundation Parameters

Based on the previous investigation and subsurface conditions encountered, the following table summarizes the foundation design parameters that were recommended in the previous report and the updated geotechnical reaction at SLS and factored geotechnical resistance at ULS are provided.



FOUNDATION DESIGN PARAMETERS

Foundation	Founding Elevation		Previous Working Stress Values ¹	Previous Equivalent Limit State Design Values		Limit State Design Values Updated to Current Industry Practice ²	
	(ft)	(m)	Safe Bearing Resistance (tsf)	SLS Bearing Reaction (kPa)	ULS Factored Geotechnical Resistance (kPa)	SLS Bearing Reaction (kPa)	ULS Factored Geotechnical Resistance (kPa)
West Abutment on Spread footing	744.5	226.9	2.5	240.0	360	350	525
East Abutment on Spread footing	746.5	227.5	2.5	240.0	360	350	525
Pier on Spread footing	736.5	224.5	2.0	190.0	280	300	450

Notes: 1. Working stress design values. The Ultimate Limit State design values are based on the working stress. No field verifications were made.
 2. Resistance Factor = 0.5 for shallow foundation (CFEM 4th edition).
 Assumed Factor of Safety is 3 (CFEM 4th edition).

The seismic site coefficient for the conditions at this site is 1.0 (soil profile Type 1, Canadian Highway Bridge Design Code (CHBDC) 2006 Edition, clause 4.4.6).

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

The foundation frost penetration depth at the site is 1.2 m according to OPSD 3090.101.



7. DISCUSSION

It is anticipated that the rehabilitation of the Adelaide Road Underpass will include removal of the existing expansion joints at the abutments and conversion to semi-integral deck ends. It will be done in 2 stages and roadway protection will be required during conversion to semi-integral abutments.

A temporary support system will be required for the rehabilitation of the underpass structure and the construction for temporary support system should conform to OPSS 404 and 539. A performance level of 2 for the protection system, according to OPSS 539, should be adopted to prevent excessive lateral and/or vertical movement of the existing embankment during construction. The contractor is responsible for the selection, detailed design and performance of the roadway protection scheme. The contractor should monitor the movement of the roadway protection system.

Groundwater control may be required during the rehabilitation work. The observed groundwater level was 1.2 to 1.5 below the ground level at the time of the previous investigation, varying between elevations 230.4 to 230.8. It is anticipated that up to 3.0 m of cut will be made at the abutment locations for the rehabilitation work and that conventional filtered sump pumping techniques will be sufficient to control seepage of water into the excavation. Groundwater control of excavations is the Contractor's responsibility.

It should be noted that the groundwater levels are subject to seasonal fluctuations and precipitation patterns.

The faces of the adjacent slopes of the abutments should be rehabilitated with rock protection, rip-rap or equivalent materials to mitigate erosion effects. Rock protection is also recommended to be placed around a 1.0 m area around the centre pier. The aggregate materials should conform to OPSS.PROV 1004 and the construction of the rock protection, rip-rap or equivalent should conform to OPSS 511.



8. CLOSURE

This technical memorandum was prepared by Mr. N. Rahman, P.Eng with the assistance of Mr. M. Khorsand, EIT and was reviewed by Mr. R. Ng, PhD, P.Eng. Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact conducted an independent review of the report.

We trust this memo is sufficient for your immediate needs. Please, do not hesitate to contact us if you have any inquiries and/or comments.

Yours truly,

Peto MacCallum Ltd.



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MTO Designated Principal Contact

NR/RN/BRG:nr-mi

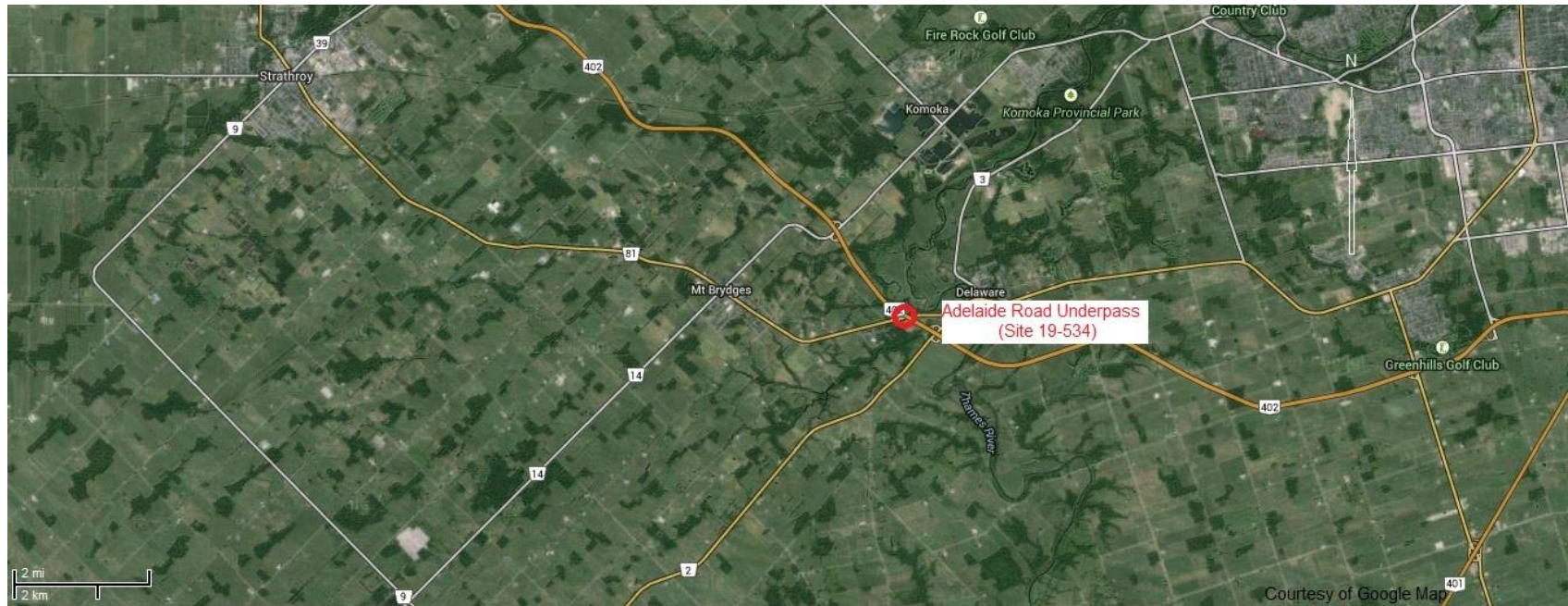


TABLE 1

LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE
OPSS 404	Construction Specification for Support Systems
OPSS 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 1004	Material Specification for Aggregates - Miscellaneous
OPSD 3090.101	Foundation Frost Depth for Southern Ontario

Figure 1 – Key Plan





APPENDIX A

Previous Foundation Investigation Report (GEOCRE 40114-104)

General Layout, Highway 81 Underpass

G.I.F-30 SEPT. 1976

GEOCRES No. 40I14-104DIST. 2 REGION W.P. No. 40-66-03CONT. No. 80-77W. O. No. STR. SITE No. 19-534HWY. No. 402LOCATION Hwy 81 Underpass
0.5 mi W of Hwy 2No of PAGES -

=====

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

40114-104

GEOCRE No.

TO: A.P. Watt (2)
Regional Structural Planning Engineer
Southwestern Region, London

FROM: Soil Mechanics Section
Geotechnical Office
West Bldg.

ATTENTION:

DATE: April 8, 1976

OUR FILE REF.

IN REPLY TO

APR 13 1976

SUBJECT:

FOUNDATION INVESTIGATION REPORT

For

W.P. 40-66-03
Hwy 402 District 2 London
Hwy 81 Underpass
0.5 Miles West of Hwy 2

Attached we are forwarding to you our detailed Foundation Investigation Report on the subsoil conditions existing at the above mentioned site.

We believe that the factual data and recommendations contained therein will prove adequate for your requirements. Should additional information be required, please do not hesitate to contact our Office.

K.G. Selby

K.G. Selby
Supervising Engineer

KGS/bp

cc: R.S. Pillar
C.S. Grebski
B.J. Giroux
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A. Wittenberg
J.R. Roy
D.P. Collins
R. Hore
J. Anderson)
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Files

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FOUNDATION INVESTIGATION REPORT

For

402 W.P. 40-66-03
Hwy 81 District 2 London
Hwy 81 Underpass
0.5 Miles West of Hwy 2

1. INTRODUCTION

This report contains the results of a foundation investigation carried out at the following site:

Hwy 81 Underpass

W.P. 40-66-03, Site: 0.7 miles west of jct.

of Hwy 2 and Hwy 81

Hwy 402 District 2, London, Ont.

The report contains factual and interpreted subsurface data and recommendations pertaining to the design and construction of the proposed structure and roadway cut.

2. SITE DESCRIPTION

The proposed underpass structure is located at the crossing of the existing Hwy 81 and future Hwy 402 on Lot 22, Conc. 1, and Lot 22, Range 1, Township of Caradoc, County of Middlesex.

In terms of topography, the immediate area of the site is flat and gently sloping towards the valley of the meandering Thames River which is located about 1200 ft. east northeast and about 1.2 miles in the southwestern direction from the proposed Jct. of Hwy 81 and 402. The river water level was found to be at elev. 665 ± in the month of May 1975. The land, in most part, is used for agricultural purposes. Some residential buildings are located along Hwy 81. These houses are supplied with adequate and good quality ground-water from dug wells or from deep wells.

Physiographically, the site is situated in the region referred to as the Caradoc Sand Plains. Irregularly placed beds or zones of clays, silts

and fine sands were deposited by a succession of glacial spillways and inter-glacial lakes.

3. FIELD AND LABORATORY INVESTIGATION

A total of four boreholes, three accompanied by dynamic cone penetration test, were carried out during the course of the field investigation. (Dec. 1 - 12, 1975). In addition, four shallow holes, to depths of 5, 10, 15 and 20 ft. were drilled for groundwater level observation.

The borings were advanced by means of a truck mounted continuous flight auger machine. When advancement in a borehole was prevented due to 'cave-in', the hole was cased with BX size casings and washboring methods were employed.

'Disturbed' samples were obtained in 2" O.D. split-spoon samplers, which were hammered into the soil in accordance with the specifications for the Standard Penetration Test. The same energy was used to carry out the dynamic cone penetration tests. 'Undisturbed' samples were recovered using 2" I.D. Shelby Tubes, which were pushed into the soil hydraulically.

The groundwater conditions across the site were determined by recording the water level in the open holes during the course of the field investigation.

The locations and elevations of all borings were surveyed by personnel from the Southwestern Region Engineering Surveys and are shown on Drawing 406603-A.

The samples were subjected to visual examination in the field and subsequently in the laboratory.

Laboratory tests were performed on selected samples to determine the physical properties of the various soil types, namely:

- Natural Moisture Content
- Grain-size Distributions
- Atterberg Limits (cohesive soils only)
- Undrained Shear Strength (unconfined)
- Bulk Density

The results of the field and laboratory tests are plotted on the Record of Borehole Sheets contained in the Appendix of this report.

4. SUBSURFACE CONDITIONS

4.1 Soil Conditions

The subsoil at the site was found to vary in both horizontal and vertical directions. Below a shallow (up to 10 ft. thick) surficial deposit of sand with some silt and traces of clay, zones or layers of cohesive and non-cohesive materials were encountered in the boreholes.

The cohesive portion of the subsoil consists mainly of clayey silt with irregular layers, seams and pockets of sand and silt. Most of these sand and silt layers were found to be water bearing. The consistency ranges from stiff to hard.

The non-cohesive deposits were found to consist of sands and silts in with varied percentage combinations. Layers, seams and pockets of clayey silt material were found interbedded within the main stratum in random occurrence. Traces of gravel were also encountered. The relative density is estimated to range from compact to dense.

Two boreholes (1 and 8) were advanced over 100 ft. in depth. These borings penetrated into a hard heterogeneous mixture of gravel sand, silt and clay (glacial till) stratum at elev. 645 \pm ft. and elev. 650 \pm respectively.

References should be made to the Record of Borehole Sheets and Drawing 406603-A for physical properties and boundaries of the different deposits at each boring location.

It is pointed out that the subsoil conditions have been established only at the borehole locations and were found to differ from borehole to borehole.

A Geotechnical Report by Hydrology Consultants Ltd., indicates that the bedrock surface is located some 225 ft. (elev. 535 \pm) below ground level in this area.

4.2 Groundwater Conditions

The following groundwater levels were observed during the field investigation:

B.H. #1	Elev. 757.0
#2	757.0
#3	756.0
#4	756.1

B.H. #5	Elev. 756.8
#6	756.7
#7	757.2
#8	756.1

The average natural ground surface is at elev. 761 \pm in the vicinity of the site.

It is pointed out that the water level observations were carried out during a relatively dry period and that higher levels will probably prevail in a wet period such as spring time.

It was observed that the non-cohesive layers (seams) interbedded within the cohesive deposits, are water bearing. Seepage from these layers are anticipated.

A 'hydrogeological study in the vicinity of the proposed highway 402 route through Lot 22, Concession 1 and Lot 22, Range 1 north Township of Caradoc' was carried out upon the request of MTC's southwestern region by Hydrology Consultants Limited (Mississauga).

5. DISCUSSION AND RECOMMENDATIONS

5.1 General

It is proposed to build a two-span underpass structure either with 163.4 - 163.4 ft. long, or 188.4 - 183.4 ft. long spans at this location. (Ref. Dwg. 406603-A)

The profile grade of future Hwy 402 (WBL and EBL) will be at elev. 740 \pm , which is about 22 ft. lower than the profile grade of Hwy 81.

The encountered subsoil, as described previously was found to be rather complex as far as the type, consistency (Denseness) and extent (vertical and horizontal) is concerned.

The observed groundwater level varied between elev. 756 \pm and 757 \pm which is 4 to 5 ft. below existing ground level.

5.2 Foundations

Two types of foundations are being considered for the proposed structure support: Spread footings and piles.

5.2.1 Spread Footings

5.2.1.1 Abutments

Assuming that the foundation level will be at elev. 748 ± a safe design load of 2.5 t.s.f. may be used for design purposes.

The subsoil at this elevation consists of stiff cohesive (clayey silt) material interbedded with sands and silts. The undrained shear strength of the cohesive portion ranges from 2000 to 4000 p.s.f. However, the base of the footing excavations could contain water bearing sand and silt layers which are sensitive to disturbance. In addition to decrease in bearing capacity, dewatering might present a problem.

In view of these facts it is recommended that before excavation is carried out interlocking sheet piles be driven to a distance equal to the hydrostatic head existing above the footing excavation base at the time of the construction.

The underside of the footing should be placed within the cohesive portion of the subsoil. Consequently all non-cohesive material should be removed from the excavation base.

The front face of the abutment footing (measured in the plane of the underside of footing) should not be placed closer than 10 ft. from the forward slope surface.

For computation of sliding resistance, an adhesion value of 3000 p.s.f. or a coefficient of 0.35, whichever results in the lesser resisting force, assumed to apply between bases of footings and the underlying soil at the foundation level.

5.2.1.2 Pier

The subsurface conditions were found to be somewhat similar to those existing at the abutments locations. The recommendations of 5.2.1.1 (with a few exceptions) are applicable to the pier design and construction.

The exceptions are as follows:

- a) Safe Design Load: 2.0 t.s.f. at elev. 734 ±.
- b) Adhesion value : 2000 p.s.f. or a coefficient of 0.35

5.2.2 Pile Support

The entire structure may be supported on one of the following pile

types:

a) Timber Piles

For #14 treated timber piles driven about 45 ft. into the original ground, a design load of 25 tons per pile is recommended.

b) 'H' Piles

End-bearing steel 'H' piles would have to be driven beyond elev. 640 in order to achieve the maximum allowable load for the respective pile section selected.

The pile driving during construction should be controlled by employing the Hiley Dynamic Pile Driving Formula. (MTC Standard SS3-10 and 11).

5.2.3 Frost Protection

The base of spread footings and the pile caps should be protected against frost action with a minimum of 4 ft. of earth cover.

5.2.4 Settlement

Total settlements under the footings should not be more than 1 inch. Differential settlements between piers and abutments should be less than 1 inch.

5.3 Roadway Cut

The proposed new Hwy 402 will be located in an approx. 22 ft. deep cut at this site. The observed groundwater level is about 16-17 ft. above the proposed profile grade of Hwy 402. The subsoil in which the roadway cut will be carried out consists of randomly varied zones or layers of stratified cohesive and non-cohesive deposits. The non-cohesive layers and seams interbedded within the cohesive zones are water-bearing. Since these pervious sand and silt layers are confined within the relatively impervious cohesive deposit, they would act as small reservoirs thus softening the surrounding soil. The trapped water would freeze during the winter period.

Upon thawing in the spring, the stored water could create large seepage forces on the face of the slope, which tend to cause surficial

instability. To ensure the longterm stability of the cut slopes the following treatment should be carried out:

- a) Perforated subdrains should be constructed at the toe of the slope. The depth of the drain should be governed by the depth of frost penetration which is about 4 ft. in this area. An adequate drainage outlet should be provided for the system.
- b) An 18 inch thick granular 'A' blanket should be provided over the slope surface as shown on Fig. 1.

This treatment should be carried out on both sides of the proposed cuts.

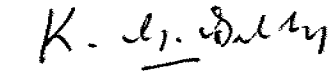
The slopes should be protected against erosion as per current MTC practices. 2:1 slopes are recommended.

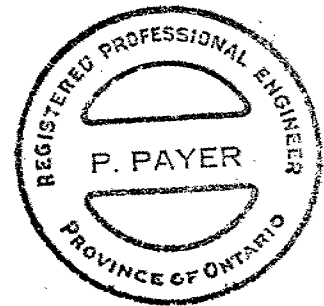
5.4 Construction Considerations

To minimize construction problems which may occur due to the observed high groundwater level, the following construction scheme is suggested:

- a) Excavate for roadway
- b) Construct subdrains and place granular 'A' blanket on slopes
- c) Construct structure


P. Payer, P. Eng.
Senior Engineer


K.G. Selby, P. Eng.
Supervising Engineer



April, 1976

APPENDIX

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 1

WP 40-66-03 LOCATION Co-ords. 15,590,243 N; 1,283,723 E. ORIGINATED BY PP
 DIST 2 HWY 402 BORING DATE December 3 to 5, 1975 COMPILED BY GP
 DATUM Geodetic BOREHOLE TYPE Cont. Flight Auger, Washbore & Cone Test CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT — W _L PLASTIC LIMIT — W _P WATER CONTENT — W			UNIT WEIGHT Y PCF	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N' VALUES		25	50	75	100	125	W _P	W	W _L		
759.5	Ground Level															
0.0	Sand, traces of silt and clay.		1	SS	9											
	Loose to Compact		2	SS	28											0 93 (7)
751.4			3	SS	34											
8.1	Clayey silt, pockets & seams of sand. Hard		4	SS	47	750										0 7 68 25
745.0			5	SS	31											5 92 (3)
14.5	Silty sand, traces of gravel		6	SS	15											
			7	SS	17											
			8	SS	26	740										0 25 45 30
			9	SS	21											
			10	SS	18											
	irregular layers, seams and pockets of clayey silt.		11	SS	16											
			12	SS	25	730										0 44 38 18
			13	SS	30											
			14	SS	31											
	Compact to Dense		15	SS	41	720										
			16	SS	11											
			17	SS	35											
711.0						710										
48.5	Clayey silt		18	SS	16											
	irregular layers, seams & pockets of sand and silt.		19	SS	22	700										
			20	SS	32	690										
	Very Stiff to Hard															
			21	SS	28	680										
			22	SS	39	670										
			23	SS	45	660										
655.5																
104.0																

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 1 Continued

WP 40-66-03

LOCATION Co-ords. 15,590,243 N; 1,283,723 E.

ORIGINATED BY PP

DIST 2 HWY 402

BORING DATE December 3 to 5, 1975

COMPILED BY GP

DATUM Geodetic

BOREHOLE TYPE Cont. Flight Auger, Washbore & Cone Test

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W			UNIT WEIGHT γ PCF	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		25	50	75	100	125	W_P	W	W_L		
655.5	continued															
104.0	Clayey silt, irregular layers, seams & pockets of sand and silt.															
	Very Stiff to Hard		24	SS	94	650										
644.5																
115.0	Het. mix. of gravel, sand, silt & clay															
	Glacial Till															
638.0	Hard		25	SS	36	640										
121.5	End of Borehole															

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 3

WP 40-66-03 LOCATION Co-ords. 15,590,188 N 1,283,772 E.
 DIST 2 HWY 402 BORING DATE December 1, 1975
 DATUM Geodetic BOREHOLE TYPE Cont. Flight Auger & Cone Test

ORIGINATED BY MK
 COMPILED BY GP
 CHECKED BY *[Signature]*

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT				LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ PCF	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N' VALUES		25	50	75	100	125	w_p	w	w_L	
760.0	Ground Level														
0.0	Sand, some silt, traces of clay.		1	SS	6										0 67 28 5
	Very Loose to Compact		2	SS	3										0 68 25 7
751.5			3	SS	16										4 76 (20)
8.5	Clayey silt, seams & pockets of sand.		4	SS	46	750									0 1 79 20
745.0	Hard		5	SS	38										
15.0	Silt, Compact to Dense trace of sand and clay seams.		6	SS	44										0 8 81 11
740.5			7	SS	26	740									
19.5	Clayey silt, occasional seams & pockets of sand.		8	SS	14										
			9	SS	16										
733.0	Stiff to Very Stiff		10	SS	23										
27.0	Silty sand to sandy silt, seams & pockets of clay.		11	SS	18	730									0 32 48 20
			12	SS	34										
			13	SS	40										
	Compact to Dense		14	SS	10	720									
713.5			15	SS	22										0 59 35 6
46.5	End of Borehole														

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 7

WP 40-66-03 LOCATION Co-ords. 15,590,205 N; 1,283,533 E. ORIGINATED BY RVV/PP
DIST 2 HWY 402 BORING DATE December 10 to 12, 1975 COMPILED BY GP
DATUM Geodetic BOREHOLE TYPE Washbore - BX Casing and Cone Test CHECKED BY *GP*

SOIL PROFILE		STRAT. PLOT	SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ PCF	REMARKS			
ELEV DEPTH	DESCRIPTION		NUMBER	TYPE	N' VALUES		25	50	75	100	125	w_p	w	w_L		GR	SA	SI	CL
761.1	Ground Level																		
0.0	Sand, some silt, traces of clay.		1	SS	7	760													
			2	SS	24														
			3	SS	111														
			4	SS	71														
751.1	Loose to Very Dense		5	SS	46														
10.0	Clayey Silt		6	SS	27	750													
			7	SS	39														
			8	TW	PH														
			9	SS	35														
	irregular layers, seams & pockets of sand and silt.		10	SS	22														
			11	TW	PH														
			12	TW	PH														
			13	TW	PH														
			14	TW	PH														
			15	TW	PH														
			16	TW	PH														
			17	SS	24														
	Stiff to Hard		18	SS	26														
			19	SS	29														
			20	SS	18														
719.6			21	SS	14	720													
41.5	End of Borehole																		

RECORD OF BOREHOLE NO 8

WP 40-66-03 LOCATION Co-ord. 15,590,161 N; 1,283,407 E. ORIGINATED BY MK/BVY
 DIST 2 HWY 402 BORING DATE December 8 to 10, 1975 COMPILED BY GP
 DATUM Geodetic BOREHOLE TYPE Cont. Flight Auger, Washbore & Cone Test CHECKED BY GP

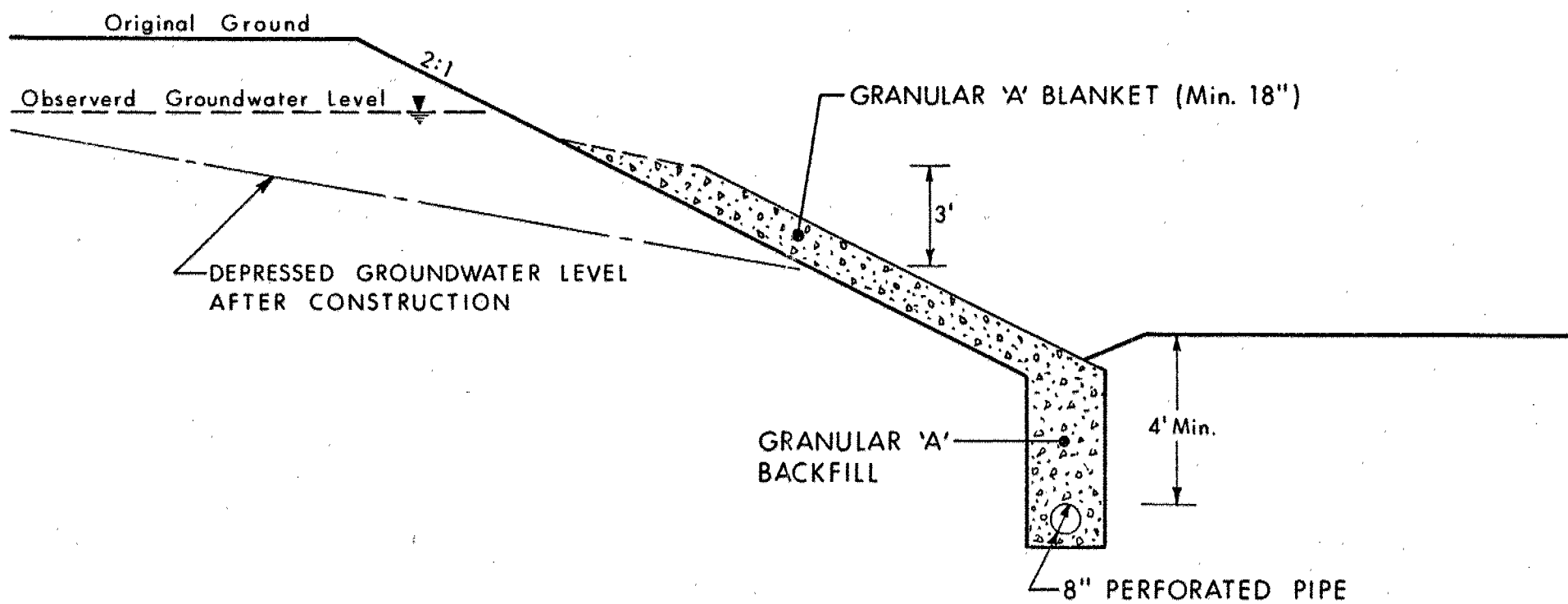
SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W			UNIT WEIGHT γ PCF	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		25	50	75	100	125	W_P	W	W_L		
762.2	Ground Level															
0.0	Sand, traces of silt and clay.		1	SS	11	760										0 87 (13)
	Compact		2	SS	19											0 86 (14)
752.7	gravel		3	SS	41											21 73 (6)
9.5	Clayey silt irregular layers, seams & pockets of sand and silt		4	SS	31											0 11 55 34
			5	SS	34											0 12 58 30
			6	SS	41											
			7	SS	28											
			8	SS	23											
	Stiff to Hard		9	SS	15											
			10	SS	23											0 4 70 26
			11	SS	29											
730.2			12	SS	21											
32.0	Silty sand to sandy silt, some clay		13	SS	30											0 47 32 21
			14	SS	23											
722.7	Compact to Dense		15	SS	32											0 18 65 17
39.5	Clayey silt layered		16	SS	15											
716.2	Very Stiff		17	SS	71											
46.0	Silt, trace of sand, irregular layers of clayey silt		18	SS	22											0 7 72 21
			19	SS	22											
	Compact to Dense		20	SS	32											0 9 66 25
696.2																
66.0	Clayey silt irregular layers, seams and pockets of sand and silt		21	SS	31											
			22	SS	81											
			23	SS	87											0 0 63 37
	Very Stiff to Hard		24	SS	29											
658.2																
104.0																

RECORD OF BOREHOLE No 8 Continued

WP 40-66-03 LOCATION Co-ords. 15,590,161 N; 1,283,407 E. ORIGINATED BY MK/BVV
 DIST 2 HWY 402 BORING DATE December 8 to 10, 1975 COMPILED BY GP
 DATUM Geodetic BOREHOLE TYPE Cont. Flight Auger, Washbore & Cone Test CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W			UNIT WEIGHT γ PCF	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N' VALUES		25	50	75	100	125	W_P	W	W_L		
658.2	continued															
104.0	clayey silt, irregular layers, seams & pockets of sand and silt.															
650.2	Very Stiff to Hard		25	SS	49	650										
112.0	Glacial Till															
645.2	Hard		26	SS	167											
117.0	End of Borehole															

RECOMMENDED CUT SLOPE TREATMENT



N.T.S.

FIG. 1

W.P. 40-66-03

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

PENETRATION RESISTANCE

'N' STANDARD PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>c LB./SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 250	VERY LOOSE	0 - 4
SOFT	250 - 500	LOOSE	4 - 10
FIRM	500 - 1000	COMPACT	10 - 30
STIFF	1000 - 2000	DENSE	30 - 50
VERY STIFF	2000 - 4000	VERY DENSE	> 50
HARD	> 4000		

TERMS TO BE USED IN DESCRIBING SOILS:-

TRACE < 10 % , SOME 10-25 % , WITH 25-40 % , > 40 % SILTY, SANDY, GRAVELLY, CLAYEY ETC.

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.T.	SLOTTED TUBE SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

U	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL	F.V.	FIELD VANE
CIU	CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL	C	CONSOLIDATION
CID	" " DRAINED "	S	SENSITIVITY
CAU	" ANISOTROPIC UNDRAINED "		
CAD	" " DRAINED "		

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_P	PLASTIC LIMIT
I_P	PLASTICITY INDEX
w_S	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX = $\frac{w - w_P}{I_P}$
I_C	CONSISTENCY INDEX = $\frac{w_L - w}{I_P}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE = $\frac{-\Delta e}{(1+e)\Delta\sigma}$
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX = $\frac{\Delta e}{\Delta \log_{10} \sigma}$
T_v	TIME FACTOR = $\frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION INTERCEPT
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
σ'	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

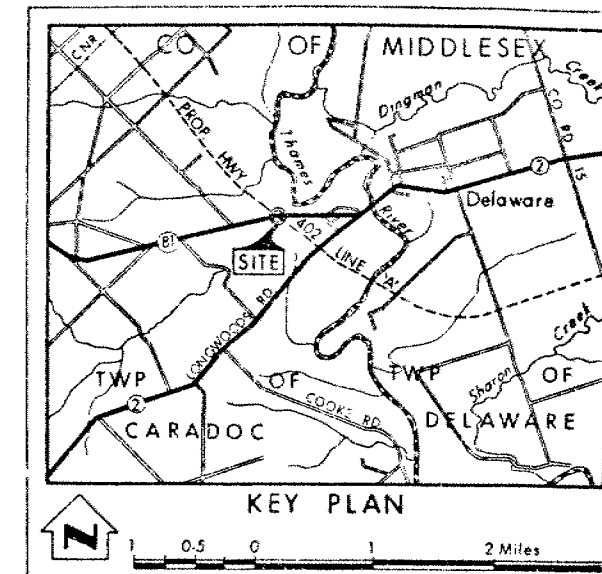
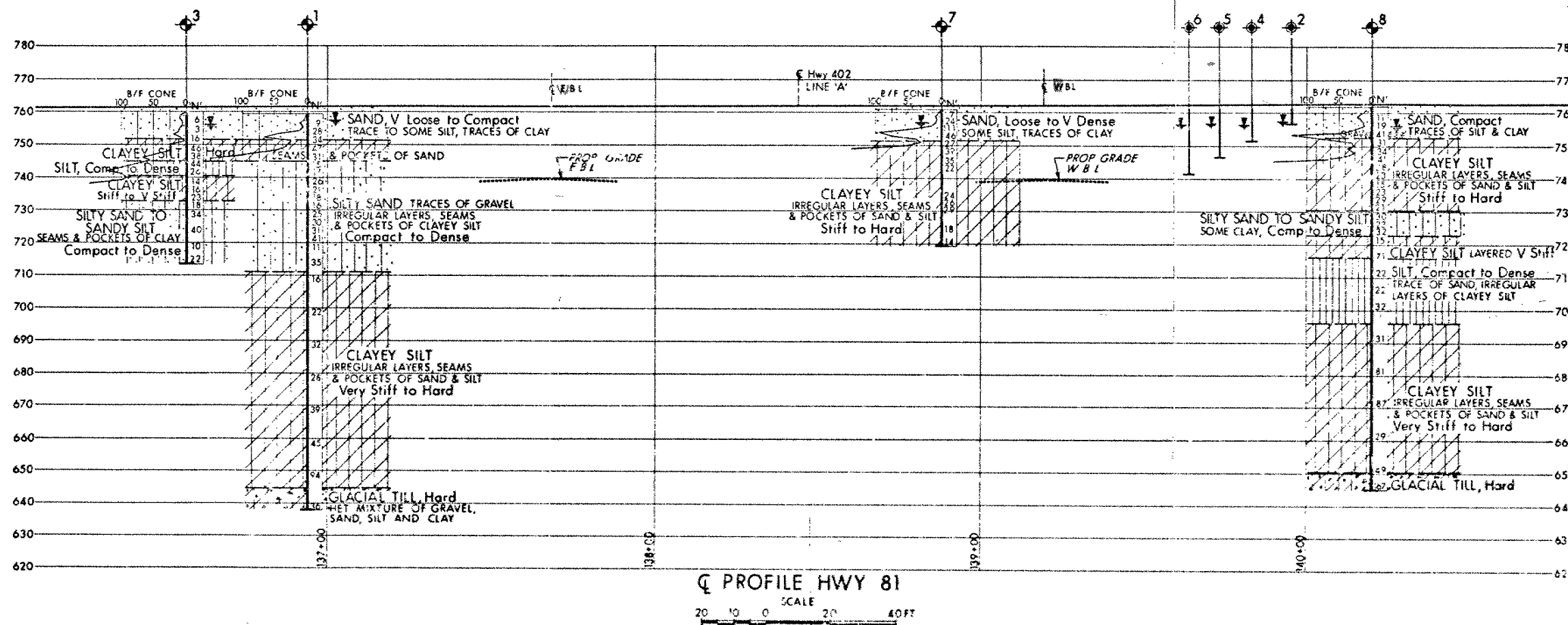
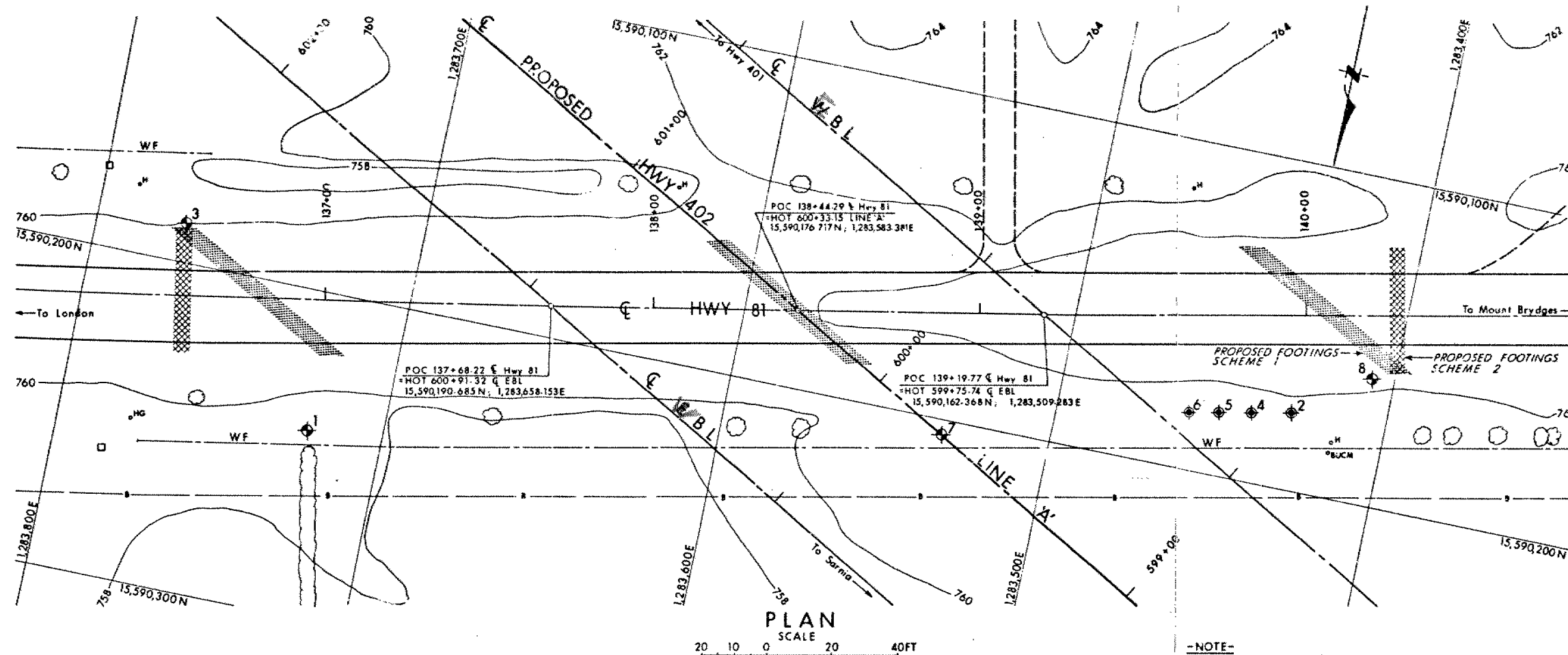
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL



LEGEND			
	Bore Hole		
	Dynamic Cone Penetration Resistance Test		
	Bore Hole & Cone Test		
	Water Levels established at time of field investigation: Dec 1975		
	Auger Hole		

NO.	ELEVATION	CO-ORDINATES NORTH	EAST
1	759.5	15,590,243	1,283,723
2	761.5	15,590,176	1,283,429
3	760.0	15,590,188	1,283,772
4	761.5	15,590,179	1,283,441
5	761.5	15,590,181	1,283,451
6	761.5	15,590,183	1,283,460
7	761.1	15,590,205	1,283,533
8	762.2	15,590,161	1,283,407

NOTE: The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO
ENGINEERING SERVICES BRANCH—GEOTECHNICAL OFFICE—SOIL MECHANICS SECTION

HIGHWAY 81
(0.5 Mile West of Hwy 2)

HIGHWAY NO. Prop. 402 LINE 'A' DIST NO. 2
CO. MIDDLESEX
TWP. CARADOC LOT 22 RANGE 1N1R

BORE HOLE LOCATIONS & SOIL STRATA

SUBMITTAL CHECKED BY: [Signature]	DATE: April 6, 1976	APPROVED: [Signature]
CHECKED BY: [Signature]	DATE: April 6, 1976	APPROVED: [Signature]
CHECKED BY: [Signature]	DATE: April 6, 1976	APPROVED: [Signature]

CRAWING NO. 406603-A
BRIDGE DRAWING NO. [Blank]

NOTES

CLASS OF CONCRETE
DECK & BARRIER WALLS ----- 5000 P.S.I.
PIER COLUMN ----- 5000 P.S.I.
REMAINDER ----- 3000 P.S.I.

CLEAR COVER ON REINFORCING STEEL
FOOTINGS, ABUTMENTS & PIER COLUMNS 2 1/2"
DECK: TOP SLAB - 2" TOP, 1 1/2" BOTT.
BOTT. SLAB - 1 1/2" TOP, 1 1/2" BOTT.
WEBS - 1 1/2"
OR AS NOTED ON THE DRAWINGS.

CONSTRUCTION NOTES

THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF 1/8".
NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED STRESSED AND BROUGHT.

REINFORCING STEEL

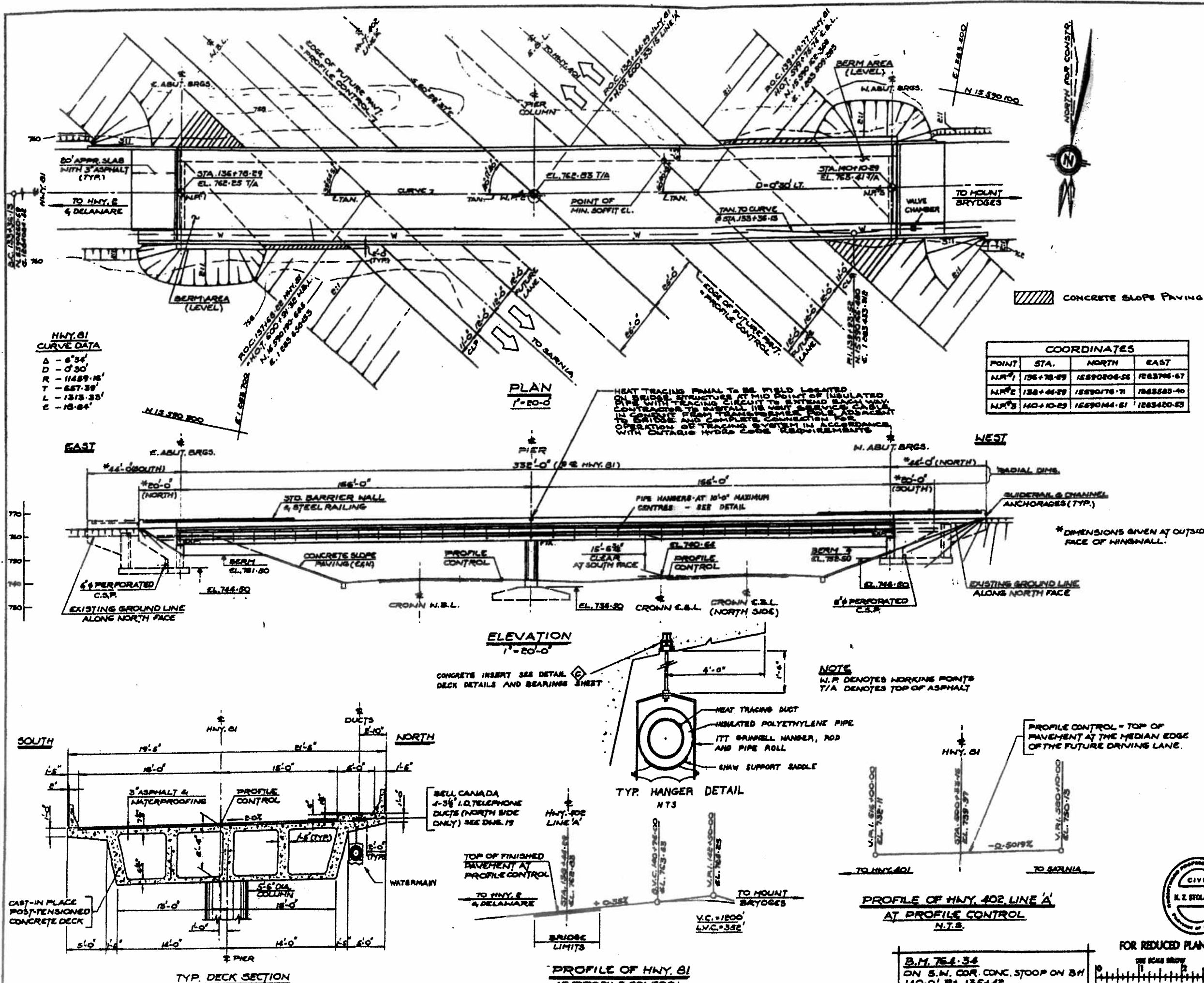
ALL STEEL GRADE 400.
REINFORCING BARS WITH THE DESIGNATION 'C' AT THE END OF BAR MARKS SHALL BE COATED BARS.
CONCRETE QUANTITIES

CONCRETE QUANTITIES ARE LISTED BELOW FOR THE APPROPRIATE CONCRETE LUMP SUM TENDER ITEMS:

CONCRETE IN PIER, ABUTMENTS, BARRIER WALLS & RETAINING WALLS ----- 12 CU. YD.
WALLS ----- 3000 P.S.I. ----- 28 CU. YD.
CONCRETE IN DECK ----- 1722 CU. YD.
CONCRETE IN BARRIER WALLS ----- 52 CU. YD.
CONCRETE IN APPROACH SLABS ----- 11 CU. YD.
CONCRETE IN SLOPE PAVING ----- 71 CU. YD.

LIST OF DRAWINGS

- 19-534-1 GENERAL LAYOUT
- 2 BORE HOLE LOCATIONS & SOIL STRATA
- 3 FOOTINGS & PIER COLUMN
- 4 EAST ABUTMENT
- 5 WEST ABUTMENT
- 6 S.E. & N.W. RETAINING WALLS
- 7 DECK DETAILS & BEARINGS
- 8 DECK REINFORCEMENT I
- 9 DECK REINFORCEMENT II
- 10 LONGITUDINAL CABLES
- 11 TRANSVERSE CABLES
- 12 BARRIER WALL
- 13 BARRIER WALL WITH SIDEWALK
- 14 STEEL RAILING (SINGLE TUBE)
- 15 20 FT. APPROACH SLAB (BARRIER WALL)
- 16 DETAILS OF CONC. SLOPE PAVING
- 17 AS CONSTRUCTED ELEV. & DIM.
- 18 STANDARD DETAILS I
- 19 STANDARD DETAILS II
- 20 STANDARD DETAILS III





APPENDIX B

Site Photographs



Photograph 1: Looking east on the west bound lane at the east abutment of structure. Slope surface adjacent to the structure was covered with snow. (February 19, 2014)



Photograph 2: Looking at the east abutment showing rip-rap stones in front the north wingwall. Weep hole observed in the wingwall (February 19, 2014).



Photograph 3: Looking at the east abutment wall. Slight soil erosion of the exposed ground was visible. Surface cracks were observed on the abutment wall. Also, presence of a weep hole was observed in the wall. (February 19, 2014)



Photograph 4: Looking at the south wingwall of the east abutment wall. Surficial cracks were visible on the wall. No soil erosion was observed of the limited exposed ground surface of the slope face. (February 19, 2014)



Photograph 5: Looking at the north wingwall of the east abutment. Surficial cracks were present on the wall. The adjacent slope surface was covered with snow. (February 19, 2014)



Photograph 6: Looking north at the center pier. Slight surficial soil erosion was observed around the pier. Minor longitudinal surficial cracks are present. (February 19, 2014)



Photograph 7: Looking at the south wingwall of the west abutment showing rip-rap stones in front the wing wall. A weep hole was observed in the wingwall (February 19, 2014)



Photograph 8: Looking at the west abutment. Slight erosion of the exposed ground in front of the abutment wall was observed. A weep hole in the wall was observed. (February 19, 2014)



Photograph 9: Looking at the north wing wall of the west abutment. Minor surficial cracks are present on the wall. The ground and the slope surface was covered with snow. (February 19, 2014)



Photograph 10: Looking at the north wing wall of west abutment wall. No slope surface erosion was visible because the slope surface was covered with snow. (February 19, 2014)